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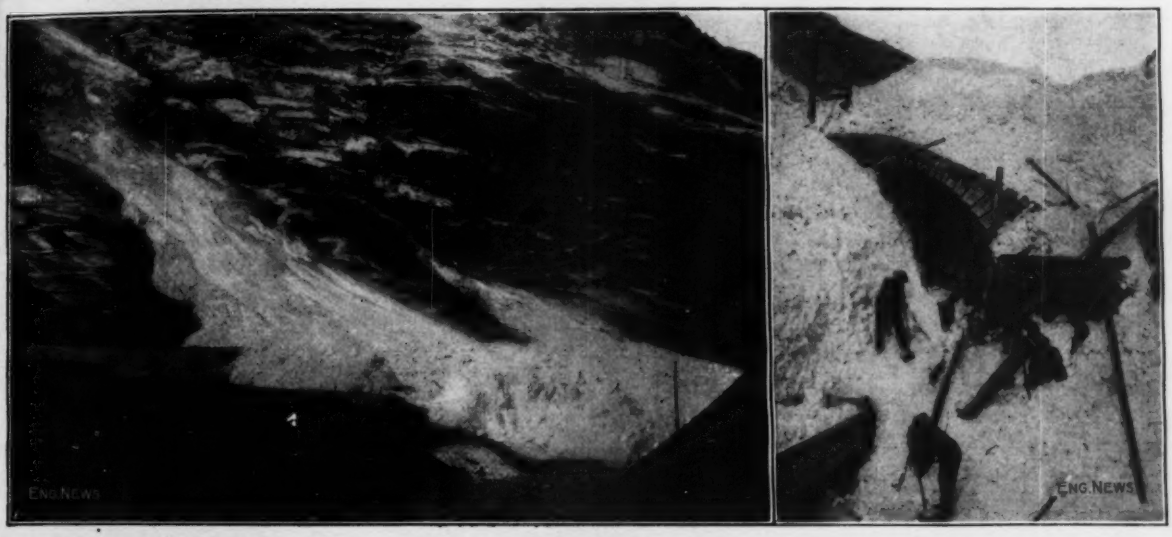
Protecting the Loetschberg Tunnel South Approach from Snowslides

The south portal of the 9-mile Loetschberg Tunnel in the Bernese Alps is exposed to dangerous avalanches. Avalanche conditions affect the entire length of the south approach lying in the Lonza Valley, and in lesser degree also the farther portion of the approach, that extending along the north slope of the Rhone Valley from Brigue to the mouth of the Lonza. On account of numerous avalanche tracks which the line crosses, many tunnels and snowsheds occur on the line. The Lonza section of the line, in fact, is almost a continuous succession of sheds and tunnels. These expensive works are responsible for the high cost of the south approach—about $280,000 per mile for the 15 miles from Goppenstein (the south portal of the tunnel) to Brigue.



FIGS. 1-2. A SNOWSLIDE COVERING THE SOUTH PORTAL OF THE LOETSCHBERG TUNNEL, SWITZERLAND, JAN. 23, 1910

In addition to the slide-protective works, slide prevention was applied in elaborate manner. The trough of the valley just at the south portal of the great tunnel was the focal point of half a dozen avalanche tracks, each permanent in character and bearing a distinctive name. Partial protection from one or two of these was secured by training-walls built over the portal of the tunnel, which deflect the slides that formerly reached the portal (Figs. 1 and 2) and pass them over the tunnel arch to the valley bottom (see Fig. 10). The others could not be dealt with by such simple means, and it was necessary to suppress the avalanches at the source by an extensive system of snow-retaining terraces and walls on the upper part of the mountain slope bordering the valley on the west.

The vital necessity of protection works for the south portal was emphasized by one of the major disasters marking the construction of the Loetschberg Tunnel. The tunnel contractors in 1907 had built an employees' restaurant just at the foot of an avalanche track, though warned of the dangerousness of the spot. On Feb. 29, 1908, after heavy snowfalls, the avalanche came down, and, though the snow itself did not strike the building, the great air-compression blast produced by the slide lifted the restaurant off its foundations and threw it down into the gorge, killing 11 and wounding 15 persons. The killed included all those occupants of the house who had been facing the hillside, while of those facing away from the slide only a few were killed, and that by being crushed under the débris.

The present Goppenstein station is nearly on the site of the destroyed restaurant, and thus is in the same avalanche track. Though protected by the snow-retaining works on the upper slope, for greater security it was built into the side of the hill, on the casemate plan, so as to pass away any slide over its roof.

SLIDE-PREVENTION WORKS

The plan to protect the south portal of the Loetschberg and Goppenstein station by snow-retaining structures and tree plantations on the mountain slope originated about 1906, but work did not begin till 1908. In the first construction period it was thought that avalanches striking just at the portal could be controlled by training-walls.

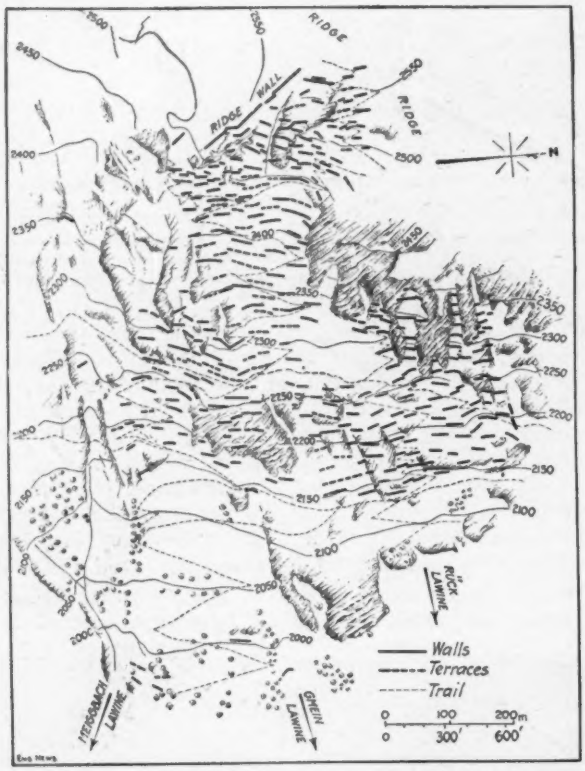


FIG. 3. PLAN OF SLIDE-PREVENTIVE WORKS ABOVE SOUTH PORTAL OF LOETSCHBERG TUNNEL

But a series of great slides on Jan. 21, 1910, which buried the whole portal (Figs. 1 and 2), showed that it was desirable to extend the retaining works northward sufficiently to reduce the amount of snow which could reach the portal. The tunnel was also extended about 175 ft. as a masonry vault, and its outer end crowned with a parapet serving as training-wall, to receive and guide the residual slides (Fig. 10).

With the northward extension of slide-preventive work just referred to, the ultimate result was that sketched in the map, Fig. 3, showing the upper mountain-slope with its terraces and walls. The physical condition of this slope is explained further by Fig. 4, a profile along one of the avalanche tracks. The ground surface involved has gradients varying from the minimum slide-starting gradient—about 25°—to about 50°, and in addition is broken by numerous precipitous cliffs. Nearly the whole area above the tree-line was steep enough to be capable of originating slides. Terrace and wall construction was applied to the upper half of this area, the part above the possible range of protective forest growth. For the lower half, resort was had to forestation.

The wall and terrace area measures about 60 acres. As the map Fig. 3 shows, a closely-spaced subdivision of this area is secured by the retaining works.

Three types of snow-retaining structure were employed. Their forms, sketched in Fig. 8, are in part a development from experience in the early part of the work and during the two winters from 1908 to 1910, in which the first-built structures suffered some damage. The layout of the works was also developed on the basis of experience. Measurements of snow depth and systematic observation of the conditions under which slides start were begun as soon as the project was definitely in hand, and continual check was thus obtained on the effectiveness of works already constructed.

Earth Terraces (Fig. 8) were used in the flattest parts of the area, i.e., on slopes of about 25°, where enough earth and vegetable growth was available to form and hold the outer face of the bank. A gutter formed at the back of the terrace, with longitudinal inclination of 5%, drains water away from the bank.

Walled Terraces (Fig. 8) were built on slopes slightly steeper than suited to earth terraces, i.e., about 30°, and on all flat slopes where an earth bank could not be held. The masonry is laid dry.

Retaining-Walls (also laid dry) were used on all steeper slopes, from 35° or less to the maximum. They have earth backing like the walled terraces, but somewhat differently arranged. These walls are the largest in size of the three classes, being built normally 6½ to 8 ft. high, and in some areas of deep drifting as much as 16½ ft. high (measured at back of wall). The top thickness is 24 in. for the low walls, and proportionately more for high walls.

Fig. 9 shows one of the walls.

In plan, the individual walls were, where possible, arched uphill, with middle ordinate ⅛ to $\frac{1}{10}$ of the length. Short wingwalls were carried back at the ends of the wall, to strengthen the ends and to retain the backing. The damage to the earlier walls during the first two winters after work began pointed to the need of strengthening the ends and the coping of the walls. This strengthening was attained by arched alignment, end wingwalls, use of the largest stones for copings, and backing the masonry with earth or stone fill.

Staggered arrangement of the walls and terraces was aimed at. The map Fig. 3 shows to what extent this was secured.

Spacing of Walls—While it was obvious that the horizontal spacing of walls in the direction of the slope should be closer the steeper the ground slope, a definite rule for

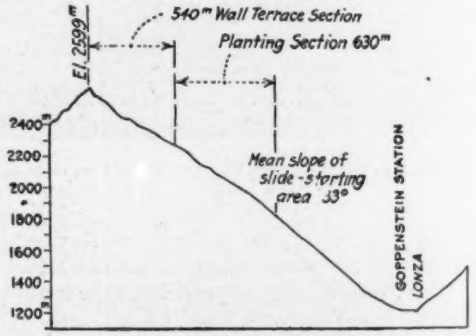
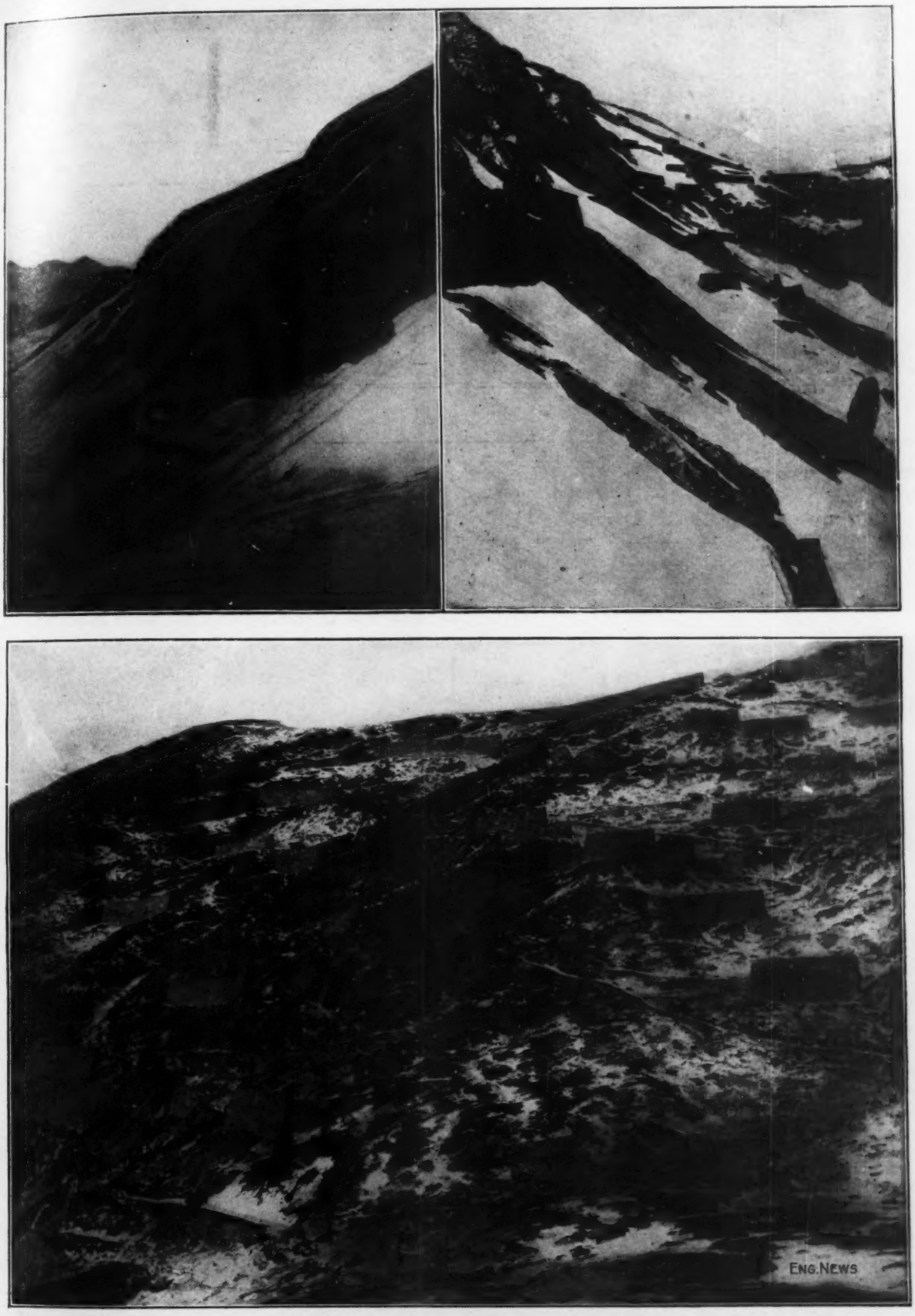


FIG. 4. PROFILE OF "GMEIN" AVALANCHE TRACK (SEE FIG. 3)

(The destruction of the contractors' restaurant building Feb. 29, 1908, killing many of the occupants, was caused by an avalanche on this track.)

determining the spacing was not available. Such a rule was developed during the work, however, after observing the behavior of the walls and the retained snow in winter. The rule so obtained is: Vertical spacing of walls should not exceed twelve times the effective height of wall (*d* in Fig. 8), except that where a terrace can be built between two walls the ratio 12 may be increased to 18.

Three views of the wall and terrace area when con-



Figs. 5-7. Slide-Protective Works Above Loetschberg Tunnel South Portal in 1910; February, June and late summer.

struction was practically completed are given in Figs. 5 to 7, two of which show snow on the mountainside. The action of the walls in holding back the snow is brought out quite clearly by these views.

Ridge Protection—One of the ridges overlooking the avalanche area is at the top of precipitous cliffs 100-200 ft. high. Under the influence of southwest winds great overhanging snowshelves formed on this ridge each winter, and their fall often gave the impetus to a general snowslide. To prevent the shelf formation a 10-ft. wall 650 ft. long was built on the ridge, 25 to 50 ft. back of the edge of the cliff. This wall succeeded in confining the drifting action on the ridge sufficiently to prevent the formation of overhang.

Treeplanting on the Lower Slope

As indicated on the map Fig. 3, the wall and terrace work extends from El. 8525 (2600 m.) down to El. 7050 (2150 m.), thus covering a vertical range of 1500 ft., and an area of 60 acres. Below this, treeplanting was done on a further area of 57 acres, down to the tree-line at El. 6230 ft. (the tree-line is lower than normal on account of the destructive action of slides). The location of the dividing line between forestation and wall construction was determined mainly by the question of the maximum elevation to which the tree growth could be extended when protected from slides. Soil conditions and climate at the site are such that this artificial tree-line could be set as high as El. 7400, but the wall work was carried down several hundred feet lower, as above noted, to give better protection until the trees have attained good growth.

Fig. 9. Snow-Retaining Wall Backed with Earth
(Some of the backing is still to be placed.)

Four varieties of tree were chosen, all native to high elevations: European larch, Norway spruce (these two making up about 70% of the total), mountain pine (*Pinus Montana*) and Swiss stone pine (*Pinus cembra*). Planting was done in clusters of four trees, each cluster at an apex of an equilateral triangle 10 ft. (3 m.) on a side. In all, 162,000 trees were used, including some 25% replacement.

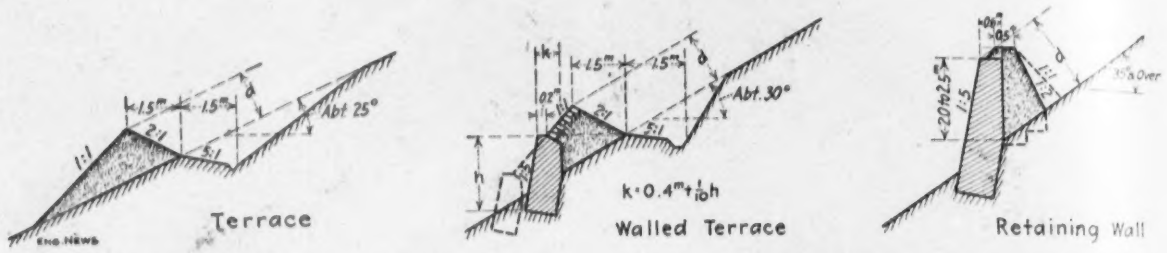


Fig. 8. Standard Sections of Terraces and Walls Used in Loetschberg Slide-Preventive Work

A barb-wire fence was built around the plantation, to protect the trees from cattle. In winter the upper and lower lines of fence are taken down, as the snow-pressure would wreck them.

Unit Prices and Cost of Slide-Preventive Works

The construction of walls and terraces and of an inspection trail, and the tree-planting, were done under unit-price contracts. The contractor had the use of a bunkhouse, but was required to supply his own tents, etc., as well as to furnish stone and other materials. The location, type and dimensions of the structures to be built were determined by the engineer as the work progressed.

The principal unit-prices paid for the work were:

Kind of work	Price per cu.yd.
Excavation in earth or loose rock, including occasional blasting	$0.28
Excavation in firm rock	0.89
Dry masonry for walls	1.55
Dry masonry for walled terraces	1.40

	Price per lin.ft.
Inspection trail (28 in. wide) in cut:	
In earth and loose work	$0.03
In firm rock	0.12

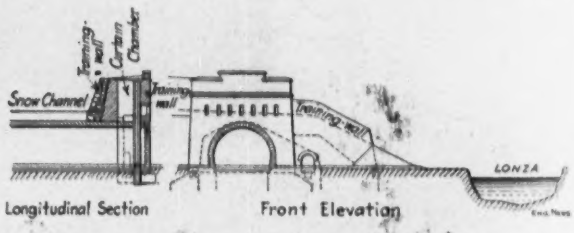


Fig. 10. Elevation and Section of South Portal

The total amount and cost of the work done on the 60-acre area was:

Kind of work	Total cost
Excavation, 23,100 cu.yd.	$10,400
Dry masonry for 300 walls and 60 walled terraces, 29,000 cu.yd.	45,000
Inspection trail, 16,400 lin.ft.	600
Total	$60,000

To this must be added the cost of acquiring the ground, which was $28 per acre.

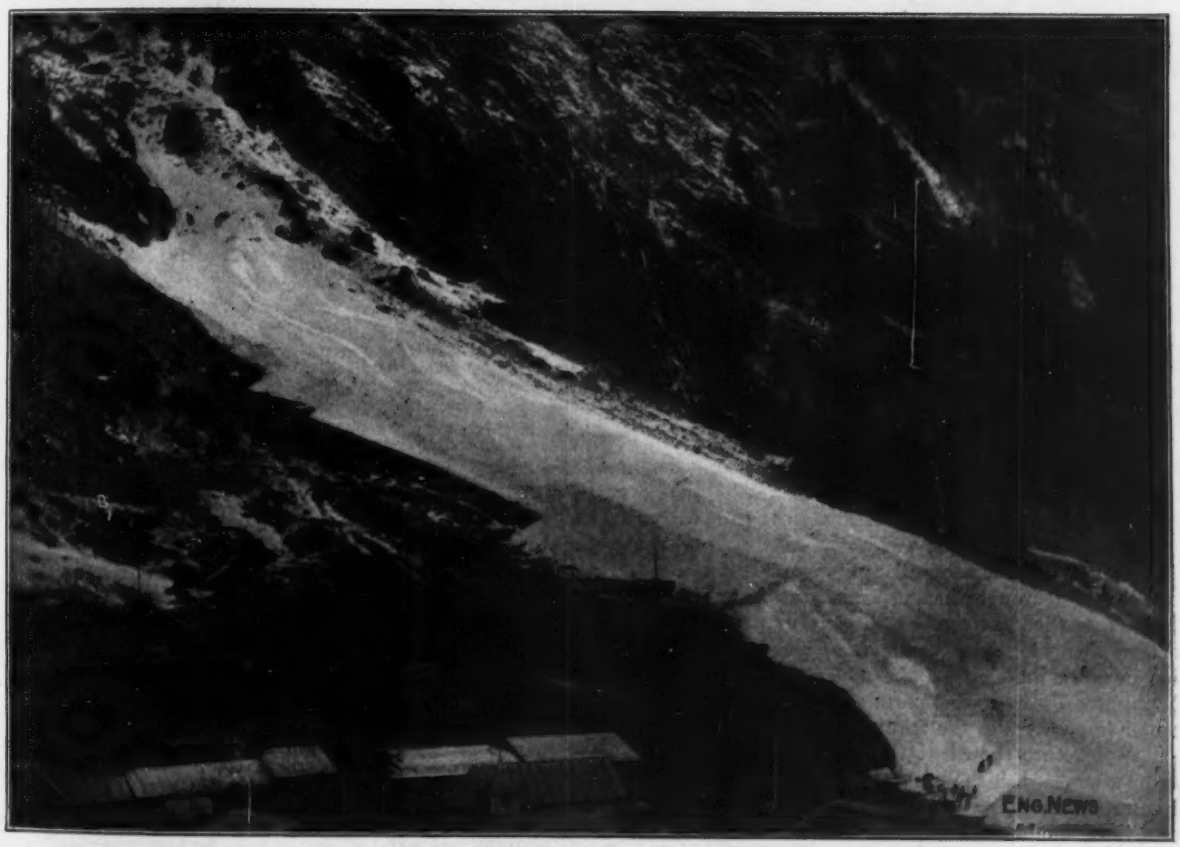


Fig. 1. (Upper View.) Slide Coming Down Over a Snowshed. Fig. 12. (Lower View.) A Longer Snowshed, Covered by the "Rotlawine" Slide.

The forestation work cost a total of $5000, made up as follows:

Item	Total cost
Cost of seedlings, @ $6 per M.	$980
Freight and haulage	400
Planting and replanting, @ $10 per M.	1620
Miscellaneous earthwork	200
Fence	1800
Total for 57 acres, 162,000 seedlings	$5000
or per acre	$88

The unit costs of similar but less extensive slide-preventive work at the north portal of the tunnel were as follows:

Wall and Terrace construction, 35 acres, $840 per acre.

Forestation, 85 acres, $60 per acre.

Line Protection by Snowsheds

The south portal of the Loetschberg tunnel is only a few feet above the level of the Lonza, but, as the valley descends very steeply, the railway line soon is high up on the side slope, in spite of its 2.7% down-grade. South of Goppenstein station, therefore, the physical conditions of difficult sidehill roadbed construction, as well as the depth of valley below the track, made tunnel and snowshed construction the most obvious means of protecting the line from snow.

A particularly bad avalanche season is represented in the two views Fig. 11 and 12, which illustrate the relation of snowslides to protection sheds. The shed of Fig. 11 is sketched in cross-sectional drawing at the right in Fig. 13. Somewhat nearer the mouth of the main tunnel is located the tunnel which is shown in Fig. 12 covered by a large slide. This tunnel passes through the débris fan of a mountainside torrent, and just above its upper portal the railway crosses the Lonza. To deflect moving snow from the upper portal and the approach to the Lonza bridge, a training-wall is built over the coping of the tunnel portal. The great slide shown in Fig. 12 taxed the capacity of the protecting works, and also blocked the Lonza on a length of 1000 ft.; the water found a way through the snow mass within two hours.

The snowshed sketched at the left in Fig. 13 is built with vaulted stone roof, and is backfilled on the uphill side to make the slope of the upper surface continuous. Training-walls over the portals confine the body of a snowslide and reduce the required length of shed.

The structure shown at the right in Fig. 13 is at a point where a bridge was originally contemplated, the alignment being far enough out from the hillside so that snowslides would remain behind and under the bridge. Later, however, it was concluded that air blasts produced by slides might endanger the safety of a train passing over the bridge at the time of a slide, and the alignment was shifted toward the hillside and protective works provided. At either side the line is in tunnel, and the open portion between the tunnels is built as a shed with steel-framed roof sheathed with 8-in. timbers. The pocket on the uphill side of the roadbed is backfilled and the fill paved on a uniform slope, as shown, and both the natural water-flow and snowslides pass over the roof. The channel for the normal drainage flow is shown by the longitudinal section in Fig. 14.

The general effect of the works described in this article is that complete snow-protection has been secured for the Loetschberg railway. Under former conditions the Lonza valley was frequently impassable or inaccessible in winter. Today it is traversed by a trunk-line railway which is intended to form the most direct and most modern trans-Alpine connection between northern and southern Europe.

The slide-preventive works, which are the most characteristic feature of this protection, were planned by Dipl.-Ing. Karl Imhof, now of Böckstein, Hohe Tauern, Austria. A description of the works was given by him before the Oesterreichische Ingenieur und Architekten Verein some time ago (see the "Zeitschrift" of the society, Dec. 13-20, 1912). For the present article we are indebted to Mr. Imhof for the loan of his original photographs.

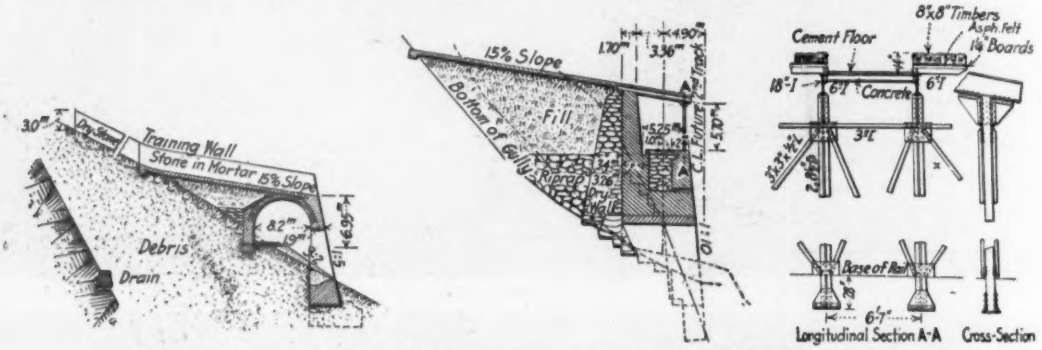


Fig. 13. Sections of Two Typical Snowsheds, Loetschberg South Approach

Reinforcement for Concrete Roads, where such reinforcement is desired or required, has customarily been placed near the top of the concrete section. In the convention of the American Concrete Institute, held in Chicago, Feb. 16-20, 1914, the subject was discussed and a number of members expressed the opinion that the reinforcement should more properly be placed near the bottom of the section, where it could act in tension when the road structure sank, as it is apt to do and to guard against damage from which the reinforcement is used. Other members argued that the road is just as apt to heave as to sink, in which case the steel would obviously be required near the top. As a compromise the specification on reinforcing was made to read: "Reinforcing metal shall be placed not nearer than 2 in. to the finished surface of the pavement."

Considerable argument was also aroused in regard to the amount of reinforcing to be used. The specification requires at least 0.038 sq.in. of metal per foot of pavement width and 0.049 sq.in. per foot of length. This is only 1/20% of reinforcement, which is absurdly low judged from structural requirements.

The advocates of the above specification stated that the steel fabric did not act in the same manner as structural reinforcement and that no method of computation could be used to ascertain the proper amount required. They did not state in what mysterious way the steel did move its wonders to perform. At any rate the specification as written, in regard to the amount of reinforcement required was allowed to stand.

Continuous-Rail Track Crossing

Railway grade crossings in main tracks where the crossing angle is about 90° are a constant source of trouble and expense to the maintenance-of-way department owing to the severe shocks caused by the wheels on each track passing the wide gaps in the crossing frogs, which gaps are necessary to permit of wheels passing on the other track. The shocks are detrimental to the wheels also, and are often felt to an unpleasant extent by passengers in the cars. Several continuous-rail crossing frogs have been designed,* the general principle being to use movable parts operated in connection with the signals and giving continuous rails for whichever track has a clear signal.

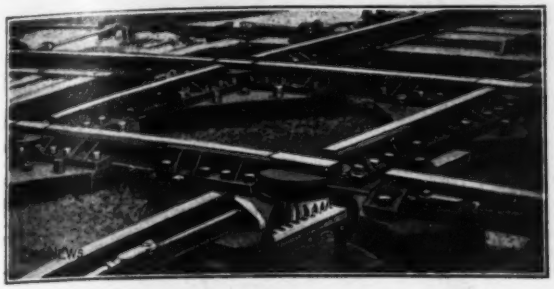


FIG. 1. TRACK CROSSING WITH CONTINUOUS RAILS
(Alexander Crossing Co., Clinton, Ill.; maker.)

The most recent of these devices is the Alexander crossing, which is now in use at two points, where it is said to be giving good results. A rigid foundation is essential to the success of any such device, and in this case an 18-in. foundation of reinforced concrete is laid under the entire crossing, with its base 3 ft. 7½ in. below the base of the rails. Upon this are laid two courses of 3-in. oak, covered with a ½-in. steel plate. Under each rail intersection is a circular or octagonal cast-steel base 7 in. deep and about 48 in. diameter, the casting being an inverted pan with oak filling on the inside.

These are held in position by vertical anchor rods built into the concrete base. The ends of the track rails rest on the castings and are bolted to heavy cast-steel splices secured in place by anchor bolts extending to the concrete base. An opening 10 in. long is left between the ends of the rails of each track.

At the center of each base casting is a 10-in. opening, to which is fitted a circular casting of manganese steel, 11½ in. deep over all. On the top of this casting is a rib corresponding in width to the rail head, while beneath it is formed a pinion, having a bearing in the base casting. On each side of one of the tracks is a heavy rack bar, sliding in guides in the base castings and having teeth which engage the pinions. In the face of the manganese-steel revolving casting are holes to receive a horizontal locking bolt.

The operating rack-bars and the locking rods are operated by pipe lines in connection with the switch and signal interlocking plants. Before setting the signals at "clear" for one track, the four revolving castings are turned so that their ribs are in line with the rails of that jecting end of one of the anchor plates on the fixed rail, which thus acts as a stop.

The first of these crossings was laid in July, 1909, at Thayer, Ill., at the crossing of the Illinois Traction System with a spur track of the C., B. & Q. R.R. There are only four or five movements daily on the steam line, while on the electric line (interurban) there are about 40 cars daily. The service, therefore, is comparatively light, but L. B. Martin, Engineer of Maintenance-of-Way, states that there is very little maintenance work beyond what is given to the rest of the interlocking plant. It is considered that it would be economical at a crossing having heavy traffic on both lines, and where the ordinary type of bolted crossing-frog construction will last only 18 to 24 months.

The second crossing was laid at Champaign, Ill., by the Illinois Central R.R., in its intersection with the Cleveland, Cincinnati, Chicago & St. Louis R.R. This was put in service Jan. 30, 1912, and is operated about 65 times per 24 hours, but the railway company states it has not been in use long enough for a comparison to be made with the ordinary type of crossing as to cost of maintenance. The expense of installation of this crossing, including foundation and signal work, was $1827; to this was added 10% of the cost of labor and 15% of the cost of material to cover the use of tools and supervision, which makes the total cost of installing $2053. This does not include the cost of the crossing, which was put in on trial. The cost of maintaining this crossing from Jan. 30, 1912 to June 1, 1913, was $29, or about $1.80 per month. The cost of maintenance from June 1 to Nov. 28, 1913, was $5, to which was added $10 for work done by the signal department in changing some crank arms and making other adjustments.

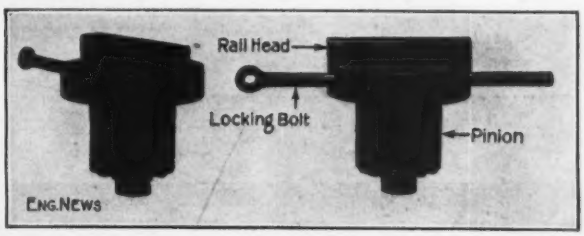


FIG. 2. THE REVOLVING FROG PIECES OF THE CONTINUOUS RAIL CROSSING

This crossing carries heavy traffic and equipment. In February, 1912, a test run on the Illinois Central R.R. was made with the track-inspection car of the University of Illinois, running over the Alexander crossing of the C., C., C. & St. L. R.R. and an ordinary crossing of the Wabash R.R. about 100 ft. distant. The diagrams of two runs show much less vibration and shock for the former than for the latter.

The continuous-rail crossing described above is manufactured by the Alexander Crossing Co., of Clinton, Ill.

*The Leighton-Hansel crossing frog of this type was described in "Engineering News," Aug. 10, 1905.

Railway Construction Reached Lake Tanganyika from the port of Dar-es-Salaam, German East Africa, on Feb. 1, completing a 780-mile line from tidewater to Kigoma, on the lake. The line is meter-gage, is now a purely governmental line, and within a year will be able to make connection via the lake and the Belgian Congo lines to the West Coast.

Contractors' Camp for Workmen on the New Quebec Bridge

By L. G. Jost*

The camp of the St. Lawrence Bridge Co., Ltd., at Quebec Bridge, is situated near the north abutment of the bridge, on a natural plateau which slopes gradually toward the east. It is protected on the north by a wooded hill and on the east and south by a low ridge of rock. As the bridge site is about half a mile from the main road, three miles from a street-car line, and eight miles from Quebec, and as the housing facilities in the neighborhood were inadequate, it was necessary to build practically a complete village. This was commenced in August, 1912, on land leased from the National Transcontinental Ry.

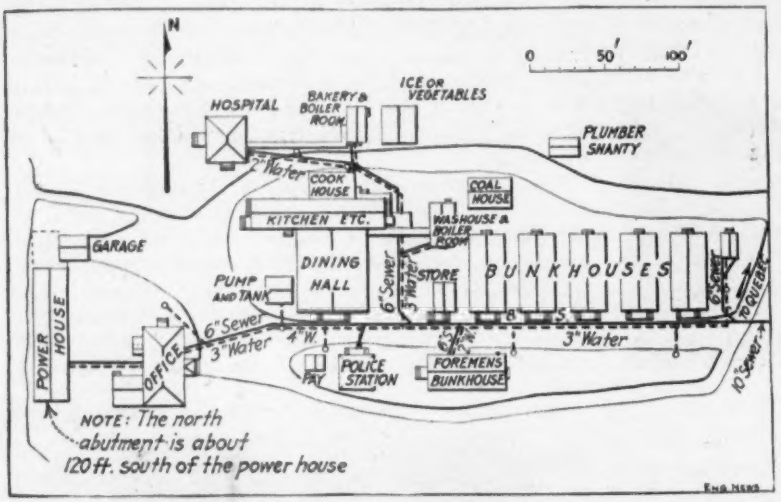


FIG. 1. LAYOUT OF CAMP OF THE ST. LAWRENCE BRIDGE CO., LTD., AT QUEBEC BRIDGE

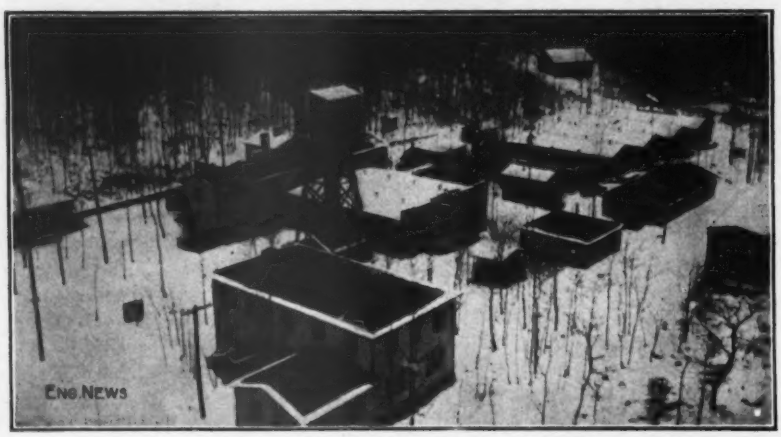


FIG. 2. GENERAL VIEW OF CAMP FROM THE TOP OF THE DERRICK TRAVELER AT THE BRIDGE

The plan (Fig. 1) gives a general layout of the camp, including the water-supply and sewerage systems. Fig. 2 is a bird's-eye photograph of the camp taken from the traveler now in the course of erection in the rear of the north abutment. Figs. 3 and 4 are interior views of the dining and the cook house, respectively.

There are five bunk houses, each 24x53 ft. in plan and comprising eight 10x10-ft. bedrooms and a 10x24-ft. sitting room in front. The inside partitions are of single sheathing 8 ft. high; each room contains two two-tier bunks. A large stove in the living room with a pipe running the full length of the building provides heat in winter. The bunk house of the foreman is similar but has nine smaller sleeping rooms and a bathroom.

The common toilet is at the east end of the camp, and is 11x18 ft. in plan with sloping concrete floor which may be hosed out. It contains two range closets; and the automatic flush tanks are kept warm in winter by coils from a jacket heater. The 17x30-ft. wash-house building contains 18 porcelain wash basins and six shower baths. At the rear is the boiler room with a low-pressure boiler from which housed pipes are run for heating the dining hall and kitchen.

The dining hall is a bright room 47x58 ft. in plan; the roof has a slope of 1 on 12, and is carried on six 15-in. 42-lb. I-beams resting on 8x8-in. posts on sides and center. It has a seating capacity for about 150 men.

The kitchen building is two stories high, the upper being used as office and bedroom for the camp steward and quarters for the culinary staff. One end of the ground floor is a storehouse for dry groceries, while the other end is a cold storage room (heretofore cooled by ice, but now by an ammonia plant).

The cooking is done in an 18x30-ft. addition at the rear. The steam cooking utensils are operated by a high-pressure boiler in a house at the rear. This boiler also supplies heat for the hospital. The latter is a 24x28-ft. building and contains an office and bedroom for the resident physician, and one ward with bathroom, etc. It is intended only to administer first-aid treatment. All serious cases will be removed as soon as possible to the Quebec hospital.

The office building is a two-story structure 28x53 ft., with a one-story wing 20x20 ft. used as a drafting room. The interior of the building is sheathed. On the second floor there are seven bedrooms, a bathroom and a living room; in a small concrete cellar, there is a low-pressure steam boiler.

Other buildings not mentioned are a garage, a small pay shack, a general store, coal house, plumber's shanty, stable, police station, and bungalows for the superintendent and the engineer in charge of construction work.

Water is supplied to the camp by an artesian well. The water is pumped to a 10,000-gal. tank on a 40-ft. tower. Five 3-in. fire hydrants take care of the entire camp. At each hydrant is a small hose house containing two 50-ft.

*Neilsonville, Que., Canada.

FIG. 3. DINING TABLE OF OFFICE STAFF

FIG. 4. INSIDE THE COOKHOUSE

lengths of fire hose. The vitrified sewer mains range from 6 to 10 in. in diameter, and with the water line are buried about 2 ft. deep and follow the natural slope of the ground, which gives a gradient from of 3.5 to 0.5 ft. per 100. At the outlet the sewage runs into a 10x10-in. wooden chute and is carried over a bluff to a point below high-water level, some thousand feet downstream from the bridge.

The camp is lighted electrically by current transmitted from Quebec. Should this supply fail, the company's power house is able to carry the lighting load.

Electric Air Drills at the Kensico Dam

The performance of electric air drills used in quarrying operations at the Kensico Dam of the Catskill Aqueduct system, at Valhalla, N. Y., was observed and recorded by the Construction Service Co. In a paper before the annual meeting of the American Institute of Mining Engineers at New York City, on Feb. 16, W. L. Saunders, 11 Broadway, New York City, described the results. The drills observed are known as the Temple-Ingersoll Type of 5-F, with cylinder 5⅝x8 in. The pulsators were driven by 5-hp. 220-volt electric motors, and full speed gave 400 strokes per minute. The bits were changed every 25 min. The drills were operated by one drill runner and a helper. The diameter of the starting bits was 3½ to 4 in. and decreased ⅛ in. for each succeeding length of steel, down to 1¾ in. The steels were from 2 ft. 6 in. to 28 ft. 6 in. in length, octagonal in section, from 1¼ to 1½ in., with square cross bits. The oil consumption of the drills was about 3 qt. per shift; the power consumption was from 30 to 40 kw.-hr. per drill shift of 8 hours.

OBSERVED TIME REQUIRED FOR THE SEVERAL OPERATIONS

Operation	No. of observations	Avg. time min.	sec.	Consumed time as per cent. of total time
Drill cutting	16	14	18	51.1
Raising drill	15	1	5	3.6
Removing bit	12	0	32	1.4
Bailing hole	11	1	23	3.4
Putting bit in hole	12	0	22	1.0
Inserting bit in chuck	16	0	23	1.4

Other operations (e.g., tightening chuck, shifting drill, etc.) consumed 38.1% of the total time.

The average cutting speed was 0.135 ft. per min. Based on these tests the following costs per foot of hole and per cubic yard of rock which was loosened by blasting, were deduced:

Item and price	Unit cost per lin.ft.	per cu.yd.
6 drillers @ $2.50	$0.1275	$0.0123
6 helpers @ $1.75,—$25.50		
Blacksmithing, $13.13	0.0657	0.0064
Total labor drilling, $38.63	$0.1932	$0.0187
Coal, 500 lb., $0.87		
Oil, 3 qt. per drill, $1.35		
Power, 35 kw.-hr. each, @ 1c	$0.0216	$0.0021
Total drilling cost, $42.95	$0.2148	$0.0208
Interest and depreciation, 2% per month, $7.70	$0.0385	$0.0037
3 powdermen @ $2		
1035 lb. dynamite @ 12c		
25 detonators @ 3c —$130.95	$0.6547	$0.0632
Total cost, $181.60	$0.9080	$0.0877

One drill cut 31 ft. in 448 min., equivalent to 200 ft. by 6 drills in 8 hours. The low cost per cubic yard is due to the wide spacing of the holes (av. 17.5x16 ft.) which were heavily charged with dynamite.

The Kensico Dam has been described in a long article which was published in ENGINEERING NEWS of Apr. 25, 1912, p. 772.

The Oldest Isolated Power Plant in a New York City office building is stated by C. M. Ripley, of New York, in a recent paper before the American Society of Heating and Ventilating Engineers, to be that in the Mills Building, Broad St., New York City, dating back to 1883 or four years after the announcement of Edison's incandescent lamp. Here are the fifteenth and nineteenth Edison dynamos built, still running with their original steam engines. These units have run every day for 31 years and are still in use furnishing regular daily service.

Plants 29 years old were reported in the Wells Building and the Dakota and the Osborne Apartments. Plants 25 years old were reported in the buildings of the Bank of New York, the Union Trust Co., and the Evelyn Apartments. One 24-year-old plant was noted in the Tower Building; one 23-year-old plant in the Nevada Apartments, and 22-year-old plants in Delmonico's at Beaver St., and the United Surety Building. An incomplete list of city plants was given as follows: 21 years' service, Presbysterian Building; 20 years, Evening Mail and St. Paul Buildings, New York Clearing House, Grace Chapel and Liederkrantz Club; 19 years, Potter, American Surety, Metropolitan, Commercial, Criminal Courts, Old Times Buildings and St. Luke's Hospital; 18 years, Bennett Building, Polhemus Memorial Clinic Dispensary, and Mechanics Bank of Brooklyn; 17 years, Waldorf-Astoria Hotel, Empire, Vincent and Sterling Buildings, Germania Bank, New York Athletic Club, National Bank of Commerce, O'Neill's Store, Terrace Garden, and Church of the Holy Trinity; 16 years, St. Paul's Methodist Church, the Ormonde and Seminole Apartments, Sprague Building, Metropolitan Museum of Arts, and Old Astor Library.

New Coast & Geodetic Survey Theodolite-Magnetometer

By Frank Waldo*

The more important nations of the world carry on magnetic observations within their own borders, and have extended these observations to their colonies and to the oceans. But it may be taken as an indication of the necessity for personal enthusiasm for the development of even a practical science that so much of the recent progress in obtaining a knowledge of the terrestrial magnetic conditions is due to the activity of Dr. L. A. Bauer, head of the department of terrestrial magnetism of the Carnegie Institution of Washington, whose magnetic surveys have been epoch making.

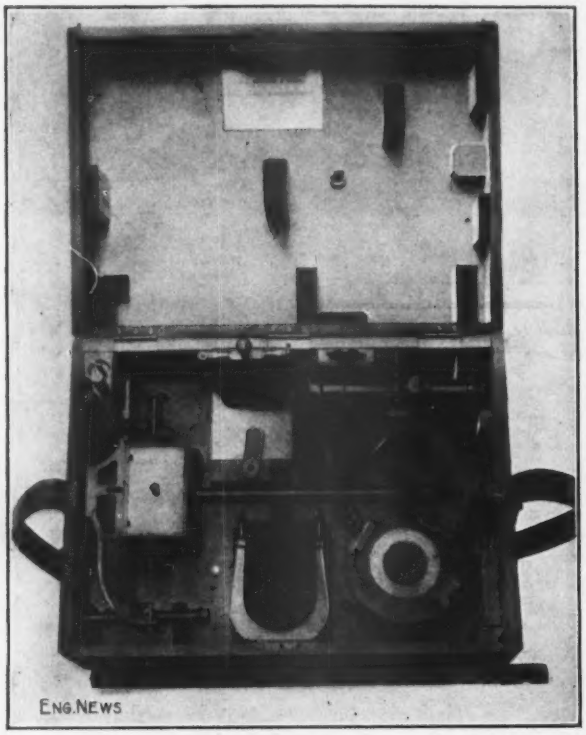


Fig. 1. New U. S. Coast & Geodetic Theodolite-Magnetometer Packed for Transport

The constantly increasing interest in magnetic data in this country is shown by the greater demand for information from the United States Coast & Geodetic Survey, notwithstanding the allied activity of the Carnegie Institution in disseminating the same information. The activity of the Coast Survey in 1911 was such that magnetic observations were made during the year at 351 stations in 332 localities scattered over 43 states and territories; and during 1912 observations were made at over 300 stations in the states and in the Philippine Islands.

The various instruments in use in different observatories give different results, no matter how well the instruments are made; and a most important part of the work of Dr. Bauer has been the unification of the observations in different parts of the world by an intercomparison of the standard magnetic apparatus of various countries and regions. The standards adopted by the Carnegie Institution have been compared with the standards of a dozen important observatories in the Northern Hemisphere and four in the Southern.

In magnetic work, it is most important that the instruments used should completely fill the requirements for the various observations to be made, which include observations for latitude, azimuth and declination, dip and oscillations of the magnetic needle. In the land survey work of the Carnegie Institution no less than half a dozen types of instruments have been employed, and some of these in several special designs. It has been a good opportunity to thresh out the adaptability of these various forms of instruments for specific uses, and to develop new designs which shall remedy defects in existing forms. The portable theodolite-magnetometer described in this article, which has been constructed for the Coast Survey by C. L. Berger & Sons, of Boston, Mass., embodies the results of the Carnegie Institute studies.

Under the instructions of Director Bauer and in consultation with him, J. A. Fleming, of the department of terrestrial magnetism of the Carnegie Institution, designed a theodolite-magnetometer with the following distinctive objects in view: portability, compactness, simplicity, minimum of loose accessories, arrangement for immediate use, and above all permitting an accuracy of observation equal to that of the best field instruments now in use.

This instrument was an adaptation of a theodolite made to order by C. L. Berger & Sons from special non-magnetic materials tested by the department; but the magnetometer parts and appliances were constructed in the workshop of the department. This instrument was found to so completely fill the requirements just mentioned, that the United States Coast & Geodetic Survey has had constructed for its traveling work a magnetometer on similar lines made throughout by Messrs. Berger & Sons.

In the following description of the theodolite-magnetometer the special points brought out by Mr. Fleming for scientific accuracy in the construction of an instrument of this pattern are presented.

There are five structural units of the instrument:

(1) The base, center and horizontal graduated circle, which forms the basis of the whole instrument;

(2) The frame work of the telescope standard;

(3) The telescope and vertical circle;

(4) The magnetometer;

(5) The tripod and support.

The instrumental parts are contained in a packing case as shown in Fig. 1. The dimensions of this case are 44 cm. long, 32 cm. wide and 17 cm. deep; and the total weight is 22 lb. The deflection bar is placed in a case of its own and carried with the tripod; making a combined weight of but 8½ lb. Thus the total weight is only about one-half the weight of the usual portable instruments.

Three leveling screws support the base. The horizontal circle, 101 mm. in diameter, is graduated to half degrees, and the two verniers read to 1′ of arc with estimations of ½′.

The standard frame for the telescope is a solid U-shaped piece, shown in Fig. 2, with two holes in the bottom to receive two pins coming up from the base, thus in-

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suring the same relative position of the parts when mounted each time for use. The frame is held by a small clamping screw-plate about 5 cm. in diameter. The clamping plate has a brass washer to prevent sticking, which is also used to clamp the magnetometer to the base when it is mounted instead of being fastened to the telescope frame.

The vertical circle and telescope are mounted by placing the axis of the telescope in the standards. The approximate leveling is done by means of the base level, reading to 1′ of arc. The stride level used for the final leveling reads to about 20″ of arc. The vertical circle which has the same diameter and graduations as the horizontal circle, is attached to the telescope, and the verniers are fixed. Cross-wires are placed in the telescope, the magnifying power of which is about 20 diameters. The instrument as used for surveying and astronomical purposes is shown in Fig. 2.

When used as a magnetometer the transit-theodolite parts are taken off. The magnetometer attachment, including the telescope, is taken bodily from the packing case and mounted on the base as shown in Fig. 3. It is to be noticed (in Fig. 3) that the magnetometer telescope, which has a magnifying power of 8 diameters, is mounted in wyes which are carried directly up from the base as extensions of the magnetometer supports, thus insuring the greatest stability. The magnetometer shelter is of wood; thus permitting observations of both oscillation and deflection, with one mounting.

In Fig. 3, the reflector is shown and the magnet is seen in position, the side door being removed to render the interior visible. The illumination apparatus and the telescope hood are made removable to meet certain observing conditions. The mirror for reflecting light has both a mirror and a white side and has both horizontal and vertical motions. The light is reflected through an iris diaphragm for more perfect light control.

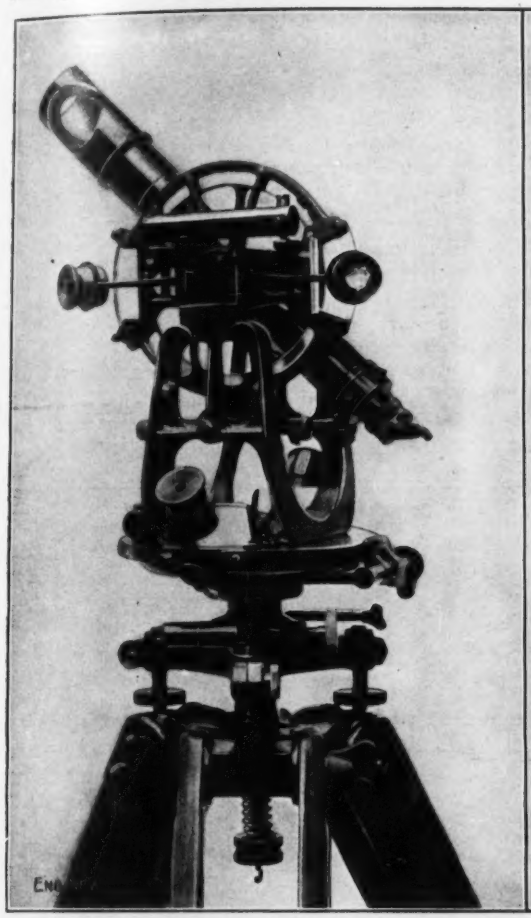


Fig. 2. Transit-Theodolite Mounted for Astronomic Observations

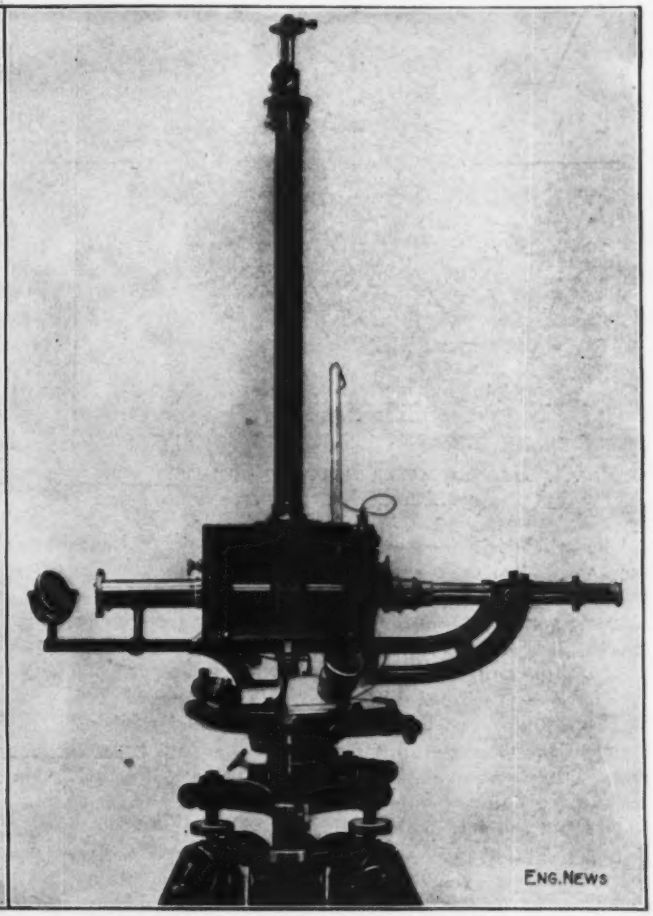


Fig. 3. Magnetometer Mounted for Magnetic Observations

The length of the suspension of the magnet is about 305 mm. The instrument mounted for deflection work with the deflecting bar in position, as seen from the eyepiece end, is shown in Fig. 4. This bar, which is 5 mm. by 15 mm. by 64 cm. in dimensions, is mounted beneath the magnet shelter.

Two pins (tapered) on each side of the wooden shelter center and hold the bar in place. The magnet is housed in a wooden box as shown in Fig. 4, together with the counterpoise weight at the other end of the bar.

The two magnets employed are of special permanent magnet steel of American manufacture and magnetized by the method given by Prof. Carl Barus. The dimensions of the magnets are as follows: Long magnet, length 56 mm., outside diameter 7.9 mm., inside diameter 5.6

mm. Short magnet, length 26 mm., outside diameter 6.5 mm., inside diameter 4.5 mm.

The two magnets are covered by gold-plated brass cylindrical sheaths which prevent rusting, and aid in mounting and balancing the collimating system; the sheaths are so made that the two magnets and sheaths weigh the same, and present the same external dimensions. In Fig. 3 the sheathed magnet is shown suspended in position: a convenient pin, groove and marks aid in the quick mounting of the magnet.

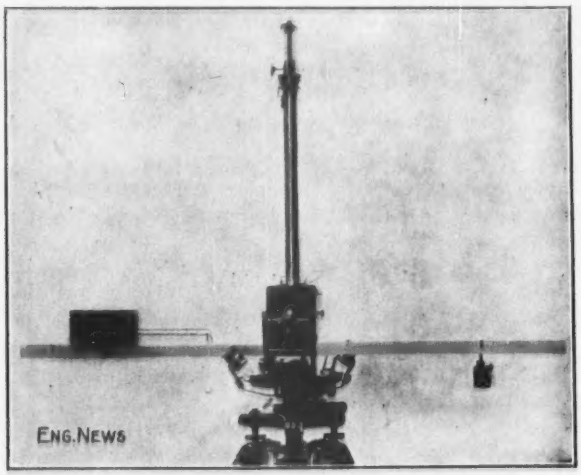


FIG. 4. MAGNETOMETER MOUNTED COMPLETE FOR DEFLECTION OBSERVATIONS

The reticle is a piece of glass with plane-parallel surfaces, with two cross lines at right angles, mounted at the south end of the sheath.

The phosphor-bronze suspension ribbon, which hangs from the suspension head shown in Fig. 3, is 0.013 mm. by 0.188 mm. in cross-section, for ordinary use, but for inertia determinations has twice this thickness. The rack and pinion just below the top, Fig. 3, is for raising and lowering the stirrup. The stirrup is of gold-plated brass to prevent oxidation. The inertia bars, 56 mm. long and 9.1 mm. in diameter, are of drawn phosphor-bronze and are gold plated.

There is an arrester consisting of two arms and a bar, for bringing the magnet to rest, and operated by a slow motion near the top of the outside of the magnet house. A scale of 60 divisions engraved on the glass reticle of the magnetometer telescope (each space being 3′ of arc) is used for referring the position of the collimating lines of the magnet to the horizontal circle.

An Automatic Headgate for Regulation of Irrigation Flow

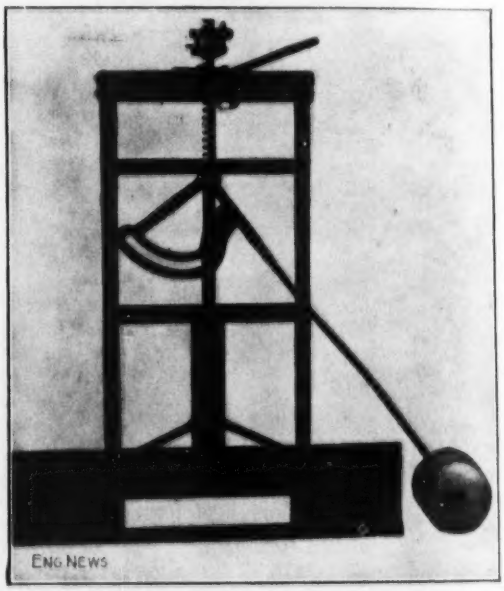
A headgate for the automatic control of irrigation flow recently tested at the Irrigation Experiment Laboratory of the Colorado Agricultural College is shown by the accompanying view. Constant flow is maintained by a change in area of a submerged orifice, corresponding to change in head, so that the product of area by velocity is constant.

Adjustment of the orifice is made by means of a vertical shutter gate, raised or lowered by a proper cam, which is in turn controlled by a cylindrical float and arm. The float has a weight slightly less than the weight of water displaced when half submerged; it is loaded with oil to bring submergence to an exact diametral plane.

The cam was designed to vary the height of orifice in exact inverse ratio to the square root of the head on the center of orifice but, for the test construction shown, was lined with babbit metal to facilitate any modification in the curve to compensate for contraction and friction under various heads. This construction would not do for a gate in continuous service and of course would not be necessary once the shape of the curve was finally adjusted for each size of gate.

Length of orifice can be adjusted to secure any desired value of constant discharge by horizontal sliding gates immediately back of the shutter gate. These are operated by a pinion, rack-lift stem and diagonal eye-bars, the latter pin connected to gates and stem. The lift rack has a micrometer lock which permits the gate being closed at any time but opened only to the point set. The stem is graduated to $\frac{1}{10}$-sec.-ft. increments, but the scale can be interpolated to hundredths.

The gate shown, as made for these tests, has a capacity of 2.61 sec.-ft.—sufficient for a farm lateral. At the Irrigation Laboratory, discharge was measured in a calibrated tank to within 0.005 cu.ft. With the cam curve as designed, the flow was constant within 5%. The gate was adjusted then so that the maximum change in flow



AUTOMATIC CONSTANT-FLOW HEADGATE FOR IRRIGATION LATERALS

was less than 2% under extreme fluctuations of head. This error was not eccentric but varied almost directly as the head. It is expected that it can be further reduced if desired.

The basic invention is that of M. I. Chapin, of Greeley, Colo.; R. W. Gelder and A. G. Lincoln, engineers, of Greeley, were engaged by the promoters, B. D. Sanborn and C. H. Ramsay, to design a simple and compact gate embodying the scheme. To Messrs. Gilder and Lincoln we are indebted for the photographs and information.

Lessons from the Sewage-Treatment Plant of Columbus, Ohio

By Rudolph Hering*

The receipt of the annual report of the Sewage Disposal Division of the City of Columbus, Ohio, for 1913, by C. B. Hoover, suggests a few remarks, in view of the facts that the Columbus sewage-disposal plant is the first large plant of its kind in this country and that the experience gained by it is, therefore, especially valuable, and most strongly emphasizes the directions in which progress is being made.

When the Columbus problem was studied in 1901, I recommended septic tanks followed by intermittent filters, because this treatment, among the several treatments then known, was the best. Caution, however, was also recommended and the advice given a year or two later that certain experimental investigations be made, in view of the economical phases of the sprinkling method of treating liquid sewage, then being developed in England. It was especially necessary to make such investigations on account of the much colder winters in our western climate. The results caused the local authorities to adopt septic tanks and sprinkling filters, which later were detailed and built.

Mr. Hoover in his last report gives a very good summary of the present condition and states his conclusions for future work.

The operation of the filters has extended over a number of years. Sprinkling filters have proved to be an economical success even in severe winter weather. At the present time it is reported that the river is seriously polluted with raw sewage and trade waste, extending for over 15 miles. Foul odors, gasification and floating sludge are present at times within a zone, the boundaries of which are determined by the dilution, the oxygen in the river water and the temperature. In consequence of this, Mr. Hoover recommends "the installation of additional and improved treatment facilities." A number of lessons may be learned from this case.

a. With a seemingly recommendable disposition, both communities and their engineers are too often inclined to curtail the cost by planning works, either with too small a margin for the natural growth of a city, or with a feeling that in the near future new solutions would require extensive alterations in any works at once to be built.

The first reason for keeping expenses down should be strongly opposed by engineers because history pretty consistently shows that unless a city outlines and persists in following a definite plan of extending its public works, they may soon become insufficient, their expenses increased and the former troubles repeated, sometimes with unfair reflections upon the engineers.

The second reason, regarding new solutions in embryo, is more difficult to handle. A city may be compelled to seek some relief at once and cannot wait any longer. The engineer must recommend a definite plan. If he has good cause to believe in the early birth of a better plan, he stands before two alternatives. Either he should at once report his belief and the grounds for it to his client, who then assumes the responsibility of action, or in the absence of such belief at the time when his recommendations are adopted, he should report any new discoveries or developments to his former clients, as soon as he is fully convinced that the art has materially improved in the designs or constructions which he had previously recommended. If engineers act conscientiously and courageously under such circumstances, respect for and confidence in the members of our profession would correspondingly rise. The responsibility for further action then again rests with the clients and not with the engineer.

b. While engineers a few years ago considered the septic tank a "necessary evil," they now generally consider it an unnecessary evil, to be replaced by some method of inoffensive sewage disposal as soon as practicable. Where no septic or ordinary settling tanks exist, the installation of the inoffensive but deeper two-story tanks is generally easy. But where large areas are already covered by the shallower septic tanks, as in Columbus and many other cities, the solution is more difficult. This at present is an instance of an expected early birth of a plan to supplement such single and shallow tanks which will secure inoffensive decomposition and sludge. It is a case where we should again exercise caution.

c. The building of final settling basins for the Columbus filter effluent did not then seem necessary to me, because the septic sludge was intended to be, and until recently was, flushed at flood times into the same river which received the filtered effluent at all times. I did not recommend such final basins for Atlanta because the effluent from the filters carries less suspended matter than the river water which was to receive it. Mr. Hoover reports the improvement of the sprinkler effluent as affected by the final settling tanks to be only 8% with respect to its stability value, which is the value to be considered for a discharge into river water not soon used for a potable supply.

Unless the effluent discharges into a stream always having clear water, I believe that money for such final settling basins can be better spent by still further oxidizing the effluent water than is usual before it leaves the filters. Nonputrescent suspended matter from a good sewage filter when entering a brook or river is no more objectionable in quality than the solid matter which is usually washed into the same by a rain storm from agricultural lands.

d. Mr. Hoover further says, that where gasification was noted in the river, sludge deposits were found, and also says that odors were never detected in the Scioto River at such points where there was any residual dissolved oxygen, and even the entire absence of oxygen was not always accompanied by odors. Similar conditions are reported from elsewhere, yet the lessons taught have apparently not yet been generally accepted.

The sewage-treatment works, according to Mr. Hoover, were in service 61% of the total time and 97% of the time from May 17 to Dec. 31. Mr. Hoover, therefore, very properly urges additional installations.

Sludge deposits in rivers cause the rapid abstraction of dissolved oxygen, even to its entire disappearance, with the subsequent condition of putrescence and of nuisance caused by the generation of sulphureted hydrogen. If there were any need for further evidence, which in my opinion there is not, the Scioto River at Columbus again clearly indicates the objectionable result of sludge deposits. We can improve many of our rivers at and be-

*Of Hering & Gregory, Consulting Engineers, 170 Broadway, New York City.

low cities discharging raw sewage into them, simply by removing the sludge deposits and getting clean bottoms, both near the shores and in the docks, as well as in the beds. I believe that in time river cleaning will be considered as desirable as street cleaning.

Columbus' experience once more verifies the opinion of many, that offensive odors do not arise from sewage-polluted rivers, if their water still contains residual oxygen, and that it is unnecessary to spend large sums of money to oxidize liquid sewage on land wherever it can be oxidized in running water in such manner that at no time and at no points the dissolved oxygen in the water will be depleted. The latter condition positively secures the absence of a nuisance, as evidenced not only at Columbus but at many other cities. The questions of water-supply, of bathing, of fish life and of agriculture, where they arise, must be answered on a different basis from that of a nuisance.

Design of a Built-In Brick Chimney

By George L. Jensen*

In the article by J. N. Jensen, Engineering News, May 9, 1912, p. 866, entitled, "The Design of a Brick Chimney," various formulas were derived and general principles deduced governing the design of a brick chimney. The article, however, dealt only with one type of chimney, namely, the completely isolated chimney. While most of the formulas derived, and all of the general principles given, for isolated chimneys, are equally true of and apply with equal force to chimneys not completely isolated, i.e., partially isolated chimneys, and built-in chimneys, nevertheless, the design of these latter types is interesting and equally deserving of attention.

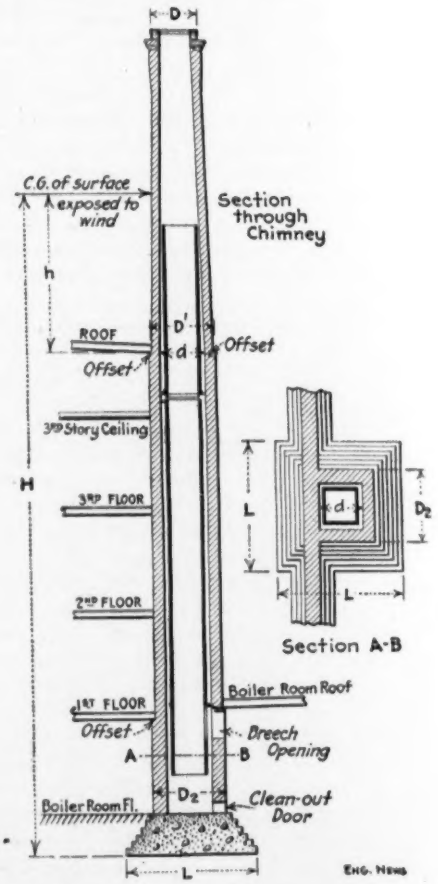
A chimney is completely isolated when its entire exterior wall surface is subject to wind pressure, and its walls are absolutely independent of any other adjoining walls of any description. A chimney is partially isolated when only a portion of its exterior wall surface is subject to wind pressure, but whose walls are absolutely independent of any other adjoining walls of any description. A good illustration of this type is a chimney that is built through a boiler-room roof independent of boiler-room walls. A chimney becomes a built-in chimney when its walls are contiguous to and continuous with other walls of any description whatever. A good illustration of this type is given in the accompanying chimney section.

A partially isolated chimney, in its design, possesses much in common with that of an isolated chimney. It differs from it only in the area of its exterior surface exposed to the wind, the corresponding center of gravity, and the length of the moment arm. This moment arm must always be taken from the center of gravity of wind surface to the section under consideration.

A built-in chimney, by virtue of the fact that its walls are continuous with the walls of the buildings surrounding it, or into which it is built, will have to be treated in many ways different from an isolated chimney. In the first place, the live- and dead-floor and roof loads which the building walls adjoining must carry will be imposed upon the built-in chimney walls, either directly or by distribution, and therefore these additional loads must be considered. In the second place, because the built-in chimney wall is continuous with the building wall proper, the entire built-in portion of the chimney becomes much more rigid and less subject to flexure than an isolated chimney, and this should be considered.

For a built-in chimney, the determining of the dead-loads and the live-loads on the masonry at the various sections and upon the soil at its foundation are simple and should require no detailed explanation. The typical section, herewith, illustrates what these particular loads might be.

In the determination of wind stresses of built-in chimneys, three possible methods of proceeding appear. All



Details of a Built-In Brick Chimney

three of these methods ignore the effect of wind on the portion of the chimney that is built-in, two of them consider the effect of the wind acting upon that portion of the chimney that is exposed to the wind, and the third ignores even this wind pressure except as to its effect upon the masonry itself at sections above the built-in portion. For one method, which we will call the first method, the moment arm of the wind is taken from the bottom of the footing to the center of gravity of surface exposed to wind, designated as H in the figure. For the second method, the moment arm of the wind is taken from the roof framing to the center of gravity of surface exposed to wind, designated as h in the figure. For the third method, this same moment arm h is used but the

*Architectural Engineer, Board of Education, Chicago, Ill.

effect of the wind moment produced is neglected as to its effect upon the footings.

By assuming the point of overturning of the chimney as at the bottom of the footing, as we do in the first method, we practically consider the chimney partially isolated and it would be designed as previously stated under that heading. But by so considering it, we would consequently be neglecting the bond of the adjoining wall to that of the chimney wall and the stiffening effect of the floor and roof construction which frame into the chimney wall and which act to materially strut or tie the chimney to the building. Of course, it would be possible to break all vertical joints between the chimney wall and the building wall and to frame all floor and roof construction around the chimney so as to actually bring about this condition, in which case, of course, the first method would be the logical one to use. But this is never done.

With the condition, then, as above stated, the actual result, if a chimney is computed by the first method, will be an excessive footing. And it is this excessive footing that makes this method apparently the safest, while it really is not. With an isolated chimney it is almost impossible to get a footing too large, but with a built-in chimney, where the chimney becomes a part of the building wall proper, and must act with it, it is apparent that if unsightly and possibly dangerous cracks in the masonry of the walls adjoining this chimney are to be avoided, it is important and imperative that the soil pressure under the chimney walls and the building walls adjoining it be equal or nearly so. It is just as dangerous to have a footing under the chimney too large in proportion to the adjoining wall footings as it is to have a pier footing of a building too large in proportion to its adjoining wall footings. Both will result in unequal settlements and cracks in the masonry.

This first method errs in neglecting the distribution of flexural stresses in the chimney walls to the adjoining walls of the building. That this distribution occurs cannot help but be apparent. The adjoining building walls with their lateral internal walls will certainly take a great proportion of these stresses as will also the roof and floor construction framing into the chimney and adjoining walls. The principal consideration is a wide distribution of the stresses, the wider this distribution the smaller the stresses per unit become, and less consideration need be given them. Ordinarily, the wind acting on the side of a building of ordinary height and width is not considered, for a good distribution of the resulting wind stresses is assumed. Why should not the corresponding stresses from a well built-in chimney be considered in the same way? Consider them so, and we will have a chimney fixed at the roof with a lever-arm from roof construction to the center of gravity of wind surface above, which brings us to the second method.

The second method, then, ignores all wind stresses on the chimney below the roof but considers stresses due to the wind above the roof. In this it is different from the third method which, in computations for footings, only considers the dead-load of the chimney plus the floor and roof loads of those portions framing into it. Many chimneys that the writer has checked up have undoubtedly been figured on this third-method basis, for by no other means could footings as small as have been put in been figured. These chimneys are still standing and are apparently in good condition. Their existence is an apparently strong argument in favor of this third method, and is an argument still used by those who still design chimneys by this method. But their standing is no indication of the factor of safety under which they exist. Nor does what has been done in the past in the design of chimneys apply to what can safely be done today, for today smoke departments of cities are demanding higher and higher chimneys. The higher the chimney, the more important do considerations of wind stresses become.

To better illustrate this second method detailed computations for wind stresses on footings will be carried through. Assume a chimney as shown in the figure. Let height of chimney from bottom of footing to top of chimney equal 120 ft., with 50 ft. of this exposed above the roof. Let D equal 6 ft., and D' equal 8 ft. Let wind equal 30 lb. per sq.ft., the usual value used for square chimneys. Then h equals 23.8 ft. The wind surface equals 350 sq.ft. Wind-load then equals 350×30 lb. $=$ 10,500 lb., and wind moment equals 10,500 lb. $\times$ 23.8 ft. $=$ 249,900 ft.-lb. With this moment, the stresses in the masonry at this section can be determined. For the purpose of this article it is not necessary to know what these stresses are. That determination has been well taken care of in the article previously quoted. It is sufficient here to know that in this section wind stresses are produced which, if the chimney is in equilibrium, will resist this wind moment. The summation of the wind stresses in each unit area of this section times their moment-arms about the neutral axis equals this wind moment. These wind stresses, of course, are in compression on the leeward side and tension on the windward. Now, what becomes of these stresses? Some small portion of them might be dissipated into the walls of the building but this is an exceedingly remote possibility. There is no escape for them except within the chimney walls themselves. We know that the dead-load of the chimney is transmitted from one course of brick work to the next lower one, taking on each increment load that each course brings with it until the footing is reached and the load is transferred to the soil below. So, also, in a similar way are these wind stresses, which reach their maximum at the roof section, carried downward to the footing. As stated before, at the roof section the resisting moment equals the wind moment. This resisting moment is then transmitted undiminished downward to the footing. At the roof section, it produces a certain extreme fiber stress, as the chimney increases in size this extreme fiber stress due to wind becomes less, but always this stress times the section modulus of the section must equal the wind moment at the roof section. This wind moment thus finally appears at the footing and from it is transmitted to the soil as were the dead-load stresses. Now, to return to the chimney assumed, let L equal 14 ft. Then section modulus of footing equals 457.3. Therefore, soil pressure due to wind equals 249,900 ft.-lb. divided by 457.3 $=$ 550 lb. per sq.ft.

Had the first method been used wind moment would have equaled 10,500 lb. $\times$ (120 $-$ 26.2) ft. equal 984,900 ft.-lb. Then soil pressure due to wind would have equaled 2200 lb. per sq.ft., or four times as much as that determined by the second method.

By the third method, as previously stated, there would have been no soil pressure at all due to wind.

A Milk Car for Electric Railways

There has been an agitation for greater utilization of the electric railway lines about Philadelphia in the transportation of farm produce ever since the present administration of the Department of Public Works, under Morris L. Cooke, took up such questions as the city markets and cost of living. Stimulated, perhaps, by the suggestions of this department, the Philadelphia Rapid Transit Co., under recent progressive management, has gone into freight service more extensively, putting in two more freight stations, enlarging a third, and putting in new sidings at two city markets.

NEW MILK CAR OF THE PHILADELPHIA RAPID TRANSIT CO.

As part of its increased service, the Rapid Transit Co. has placed a new type of milk car in service, making daily trips between Philadelphia and dairy districts to the north. It should be noted, however, that this does not mark a new departure of the company, for it has run milk cars for six years past between Philadelphia and Doylestown, 25 miles out.

The new car, shown by the accompanying figures, has a length of 36 ft. over corner posts, with 4-ft. vestibuled platforms. The side sills are of 5¼x8-in. yellow pine reinforced with 8-in. channels inside. The end sills are channels. Four center sills are formed by 7-in. I-beams running between end sills. Crossing are 4-in. I-beams with 6x6x⅜-in. sill connections at the ends, bolted to the side sills.

Two bulkheads at the center cut the interior into a cross passage and two main compartments. Along the sides of each section are folding sectional tables for the milk cans. Along the outer edge of each table is a T-bar with one leg projecting to support cross shelves. The floor is covered with galvanized iron laid on top of asbestos insulation. For cooling, the car has a 2-ft. 3-in. by 3-ft. 8-in. brine tank, ice chest, fan motor and piping. The cooled air is circulated through each compartment by the ducts shown. The car described was built by J. G. Brill Co., Philadelphia.

Water for Locomotive Purposes on the Trans-Australian Railway is to be obtained by conveying it in wooden pipes from Kalgoorlie for a distance of about 257 miles, service tanks being provided every 50 miles. In addition an underground supply has been located by a bore at 344 miles from Kalgoorlie, and from that point water will be pumped to tanks at other points along the line.

Progress in Manufacturing Vitrified Paving Brick: The Wire-Cut-Lug Brick

BY WILLIAM C. PERKINS*

The process of manufacturing brick for paving purposes has become more scientific as the demand for brick has increased and the methods of construction have improved.

Thirty years or so ago streets were paved with building brick made from material which would stand hard burning and semivitrification. These were, of course, what are known as plain wire-cut brick and of standard building-brick size. These small bricks made a smooth, durable pavement but it was difficult to separate the bricks by uniform spacing to allow for the proper penetration of the filler or bonding material.

The engineers in charge of street-paving work demanded projections on one side of the brick which would make possible the proper separation of each course so as to allow the filler to enter the joint and bond the courses together. The manufacturer, to meet this demand, repressed or reformed the brick immediately after the blank of clay was cut from the column and before drying, placing raised letters or various forms of spacing lugs on the side of the brick. The edges were also rounded or beveled, as it was not practicable to repress, satisfactorily, square-edged brick. The size of the brick was also gradually increased to that of the present standard block.

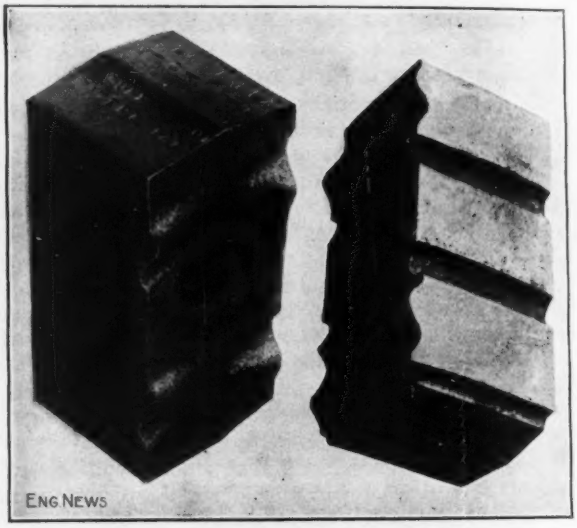


FIG. 1 FIG. 2

Fig. 1. A wire-cut lug paving block showing lugs formed on the bonding face. Fig. 2. A wire-cut lug paving block with "Hillside" grooves on the wearing face, for use on steep grades.

Repressing a wire-cut brick adds nothing of value except the lugs and even with the greatest care in manufacture many lugs are malformed or stick in the mold. In fact, the repressing is detrimental, as it changes the internal structure, makes the brick less dense and often causes extensive irregular laminations. The edges of the brick being rounded and the sides made smooth by the repressing, the cement filler does not properly adhere.

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In 1910 a wire-cut-lug brick, that is a wire-cut brick with lugs formed in the cutting, was made under the Dunn patents; and with its advent, the wire-cut brick resumed its former high place as a paving material.

For those unfamiliar with brick manufacture, it may be explained that the tempered clay is forced out of the auger machine in a long continuous column about 9¼ in. wide, and 4¼ in. thick. A cutter passes through this column of clay, cutting off successive bricks and forming the lugs on each brick at the same time. The cutting is done by wires passed transversely through the column, guided by slots in uniformly spaced steel plates. Offsets in these slots deflect the wires so that the bricks and lugs are cut uniformly.

By an ingenious arrangement of the deflecting points in the slots on the upper and lower steel plates, two separate lugs are formed on each edge of the brick giving it four points of contact with its fellow when laid in the pavement. This provides for the horizontal flow of the bonding material between the upper and lower lugs, insuring a perfectly grouted joint.

All standard wire-cut-lug brick have $\frac{3}{16}$-in. lugs and double-beveled ends (the bevel being $\frac{1}{16}$ in. from the vertical). This double bevel allows for proper penetration

FIG. 3. SECTION OF PAVEMENT LAID WITH HILLSIDE GROOVED-WIRE, CUT-LUG BLOCKS

of the filler at the ends of the brick, the adjoining bricks theoretically touching on a line at the center with at least a ⅛-in. opening at top and bottom.

The principal advantages claimed for the wire-cut-lug bricks over the repressed bricks, are:

1. *Density*—In order to repress a brick the soft block coming from the auger machine is fed into a stamping machine, the mold box of which is longer and wider than the block of clay. The pressure of the plunger first spreads the clay in length and width to fill the mold and continued pressure forms the lugs or raised letters. The reforming breaks up the inner structure of the brick and often causes irregular laminations.

2. *Uniform lugs*: The brick and its lugs being cut at one operation by wires passing through slots in a steel plate, necessarily insures uniformity. Owing to this uniformity of lug and width of brick, a perfect alignment can be obtained, when laid in pavement.

3. *Bond*: The rough wire-cut surface of the brick gives a much stronger bond with the cement-grout filler than does the smooth surface of the repressed block.

4. *Surface*: The edges of the wire-cut-lug brick being square and sharp, the cement filler can be brought up flush with the surface of the pavement and will not break away under traffic as it does from the rounded edges of repressed brick. Thus the cement-filled joints and the bricks wear uniformly and the pavement is kept smooth for traffic. The most severe flushing by street cleaners cannot affect the grouted joints.

5. *Abrasion Tests*: Rattler tests made of wire-cut-lug and repressed brick of the same material and burned to the same hardness show little difference in loss. The wire-cut-lug brick must, therefore, be much stronger and denser than its repressed brother, for the action of the rattler would necessarily be more severe on the sharp, square edges of the wire-cut-lug brick.

In order to give a better foothold for horses on grades steeper than 6 to 9%, it has long been customary to bevel one edge of a repressed paving block. These bevels on the wearing edge of the bricks make a series of shallow grooves extending across the street from curb to curb. While the horses can thus catch their shoe calks, these grooves slightly roughen the pavement and tend to increase traction and in time the bricks tend to wear rounding.

To obviate these objections, a new design of hillside brick is now made by the wire-cut-lug process. Each brick has formed on its edges a series of shallow grooves about 3 in. apart. These may be of such depth as the engineer desires.

In laying these bricks in the pavement, they are laid longitudinally instead of transversely and as the bricks of successive courses break joint, the grooves are also broken at every joint. Thus a broad tire like that of an automobile wheel will be supported continuously as it is wider than the thickness of a single course of bricks. Further, a horse gets a hold for his shoe-calks almost anywhere on the pavement.

The laying of the bricks longitudinally instead of transversely, makes it much easier to roll the bricks with a steam roller and not do undue damage in passing over the curved crown of the road.

It may be objected that the bricks being laid longitudinally, instead of transversely, might possibly be loosened and rock in the pavement under heavy wheel loads. It is believed, however, that the very strong bond of each brick with the ones adjacent, due to the rough wire-cut surface, will absolutely prevent any such rocking action, and that the pavement will show much less resistance to traction than the other types of hillside brick.

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The Consulting Engineer and the Municipal Engineer—Speaking at the inaugural dinner of the Association of Consulting Engineers of Great Britain, J. H. Balfour-Browne said the association represented an attempt to keep the profession of engineering on the heights.

He could conceive no worse fate for engineers than that they should become an instrument of manufacturers or the slave of municipal corporations.

In order to maintain a profession at its highest level it should have an external conscience in addition to the individual consciences of its members, and the association, with its rules for moral guidance, was a conscience of that kind. With regard to trade and commerce the members of the association must stand in splendid isolation. New trades ought not to be handicapped by the competition of public departments operating with funds provided by the very people with whom they were competing. Further, state and municipal engineers should not be allowed to compete with what he would call ordinary consulting engineers. They should devote all their time and attention to the work of the municipality, with adequate pay, and if they did that adequately they ought to have no time for engaging in private practice. —"London Times Engineering Supplement."

Water Supply and Sewerage Systems of the U. S. Naval Training Station, Great Lakes, Ill.

The U. S. Naval Training Station, Great Lakes, Ill., was established by the United States Government and opened on July 1, 1911, for the training of young men of the middle West for service in the U. S. Navy. It is located on the shore of Lake Michigan and is about 30 miles north of Chicago. The site is about 68 ft. above the water level. The station was designed for an ultimate capacity of 2500 and an immediate capacity of 1500 men. There are 45 buildings in all on the 160 acres comprising the grounds, and they have a total content of 13,000,000 cu. ft., costing (with equipment) approximately 26½c. per cu.ft. or a total of $3,500,000. The construction of all buildings, except a few sheds, is entirely of concrete, steel and brick. Fig. 1 is a general plan.

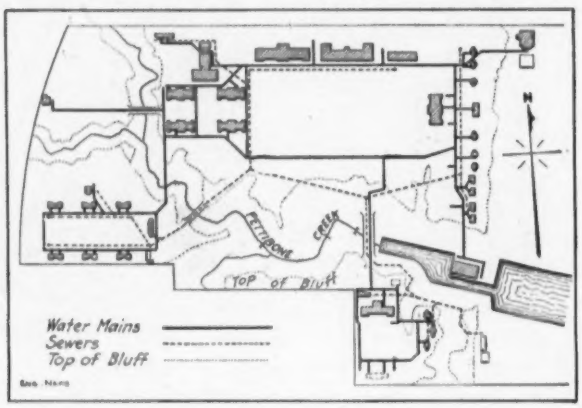

FIG. 1. PLAN OF THE U. S. NAVAL TRAINING STATION AT GREAT LAKES, ILL., SHOWING THE WATER SUPPLY AND SEWERAGE SYSTEMS

The design and installation of the water supply, sewerage and storm-water drainage systems were made under the direction of Rear Admiral Albert A. Ross, U. S. Navy, Commandant of the Station during its construction. The design was made by the late W. S. McHarg, of Chicago, and the construction was at first under Civil Engineer George S. McKay, U. S. Navy, who was succeeded by Civil Engineer C. D. Thurber, U. S. Navy. At the present time, Capt. Geo R. Clark, U. S. Navy, is the Commandant, and Lieut. E. F. Johnson, U. S. Navy, is Public Works Officer. For information in regard to the work, we are indebted to the above officers and to Lewis A. Pease, operating engineer at the station.

WATER-SUPPLY

INTAKE SYSTEM—The water-supply is taken from Lake Michigan, by a line of 14-in. cast-iron bell-and-spigot pipe which extends to a timber crib 1200 ft. from shore. The pipe has ball joints every 60 ft., and is laid so that its top is 2 ft. below the sand bottom of the lake. The end of the intake pipe in the crib extends upward and has its end open 2 ft. 6 in. above the lake bottom.

The crib, shown in Fig. 2, has its sides constructed of 3x6-in. slats, spaced 2 in. apart, securely fastened to 6x6-in. sills and framework, while the deck is made of 2x6-in. slats, spaced 2 in. apart and supported on 8x8-in. corner posts. The stone tables, which are loaded with loose rock to anchor the crib in place, are of 2x6-in. planks also fastened to the sills. All woodwork is fastened with ½-in. boat spikes and ¾-in. iron bolts with heavy cast-iron washers. On the deck is a door of slat construction, 38x42 in., with heavy brass hinges.

The crib proper is 7 ft. 10 in. by 8 ft., and 6 ft. 6 in. high, with the four sides on such an angle as to make the base on each side four feet wider, which, with the stone tables, makes the dimensions 20 ft. 9 in. by 20 ft. 4 in. The bottom of the crib is set 18 in. below the bottom of the lake, which is 6 in. over the top of the intake pipe.

The shore end of the intake pipe is in a well 10 ft. square, shown in Fig. 3. The lower part is constructed of Wakefield triple-lap sheet piling (with planks 2x10 in.) and has a concrete bottom; angle irons are used in the corners. The piling is held in place by waling timbers. The upper part of the well is of concrete supported upon the piling, and is conical in form.

The intake is so arranged with valves and fittings that water can be taken directly to the pumps without using the well, or the water may be discharged from the intake pipe at one side of a copper screen and taken by the

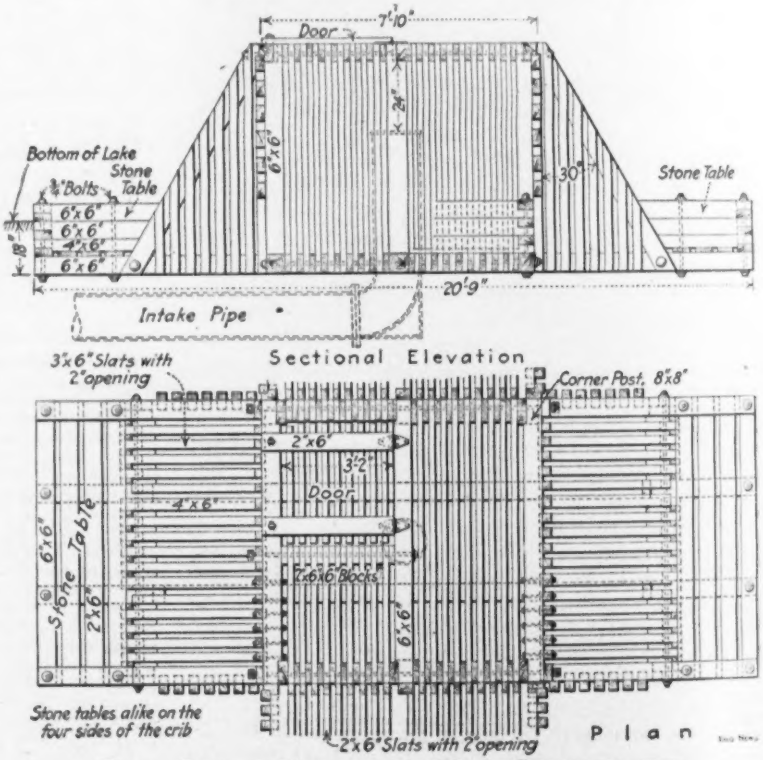

FIG. 2. INTAKE CRIB FOR THE WATER-SUPPLY OF THE U. S. NAVAL TRAINING STATION

pumps from the other side, thus screening out all fish, etc., entering the intake. There are three screens 26½x 123½ in., and two 26½x61¼ in. They are made of No. 16 copper wires, ⅝ in. apart, fastened to No. 10 copper-wire frames, which are stapled to a wood framework dividing the well. There is an operating platform for the hand-wheels of the valves, and ladders are provided so that attendants can make inspection, handle valves and clean screens when necessary.

During severe storms on the lake in the winter many of the intakes along the west shore of Lake Michigan have been blocked by ice for many hours at a time, with a total stoppage some days. Although this intake has been blocked with ice, it has never been so clogged that consumers were aware of it. When clogging occurs, the pumps are shut down and water from the 45,000-gal. tank at 75-lb. pressure is used to flush the pipe, clearing it in a few minutes.

From the intake well the water is taken in a 14-in. cast-iron main to two 100-gal. centrifugal pumps, two 750-gal. fire pumps and one 400-gal. compound service pump. The valves are so arranged that all pumps may draw directly from the intake or all piston pumps may draw from the filter and discharge into the main water system, at the same time that the two centrifugal pumps are taking water from the intake and discharging against a total head of 18 ft. to a slow sand filter. In case of necessity, the fire pumps may be used to discharge into the filter.

It has been found that instead of requiring 100 gal. per capita per day, the station actually requires approximately 300 gal. This is due to the large amount of water used in cleaning and scrubbing, and for the swimming pool, etc. This has made it necessary to install a 300-gal. centrifugal pump in place of one of the 100-gal. pumps originally installed. The total pumpage averages about 300,000 gal. per day for 1000 people.

Distribution System—From the discharge header located directly over the suction header in the pump room, a 12-in. cast-iron main is carried up the hill, 68 ft. above the lake level, to the main loop. This is a 10-in. main surrounding the main parade ground and the four dormitories at the west end of the parade ground. At the administration building, at the east end of the parade ground, is a 45,000-gal. steel tank connected to the main line with a 10-in. pipe. A valve on this line in the basement of the building permits of raising the pressure in the mains to 150 lb. in case of fire. Even under normal conditions of operation the pressure due to head from the tank would be 40 lb., and to guard against the high pressure in the buildings, reducing valves have been placed in them, keeping the pressure down to 20 lb.

The service lines are in most cases galvanized iron. In case of fire lines and lines over 2-in. diameter, cast-iron pipe is carried directly into the buildings, while in small service lines in officer's quarters, the iron is connected to lead pipe directly outside the wall. In many buildings where only small service lines are required, the fire line is separate, but where a large amount of water is required, such as in dormitories, mess hall, etc., the service and fire taps are from the one cast-iron main carried into the building.

The distribution system is as follows: 1083 ft. of 12-in. mains; 6387 ft. of 10-in. mains; 3306 ft. of 8-in. mains; 4859 ft. of 6-in. mains; 464 ft. of 4-in. mains; 50 hydrants; 54 valves.

All mains and branches are laid approximately 5 ft. deep. The mains are of cast iron, except that at three places where they cross bridges wrought-iron pipe is used, securely lagged with hair felt to prevent freezing. At each abutment there is a slip expansion joint, allowing for expansion of pipe crossing under bridge.

Water-Filtration Plant—As the sewage from the city of Waukegan is discharged into the lake only a few miles north of the training station, it is necessary to treat the water-supply. At certain times the lake water also contains considerable silt in suspension, and in order to avoid clogging the filters the water is delivered first to a circular settling basin of about 150,000 gal. capacity. The filter plant is of the slow sand type, with

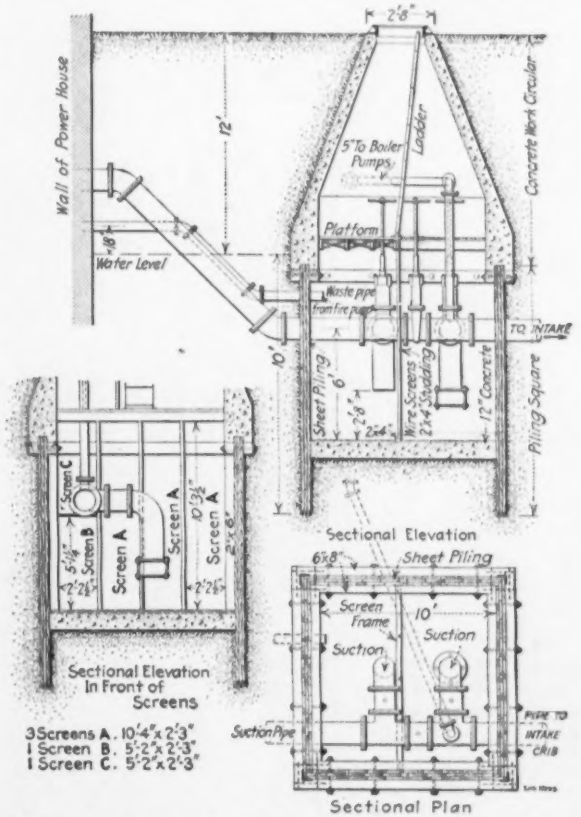

Fig. 3. Intake Well and Valve Chamber at the Shore End of the Intake Pipe

a capacity of 200,000 gal. daily. There are two filters 62x45 ft., 8 ft. deep, having a bottom 8-in. layer of gravel over the collecting tile, this being covered with sand, over which the water is usually 4 ft. deep. From the filters the water passes to a clear-water basin of 250,000 gal. capacity.

Sewerage

The sewers consist of salt-glazed, vitrified-clay sewer pipe of the hub and spigot pattern. Straight and curved pipe were furnished in 2½-ft. lengths. Branches were allowed in 2-ft. lengths. At a bridge crossing spiral-riveted steel pipe and cast-iron pipe are used, the former on the bridge and the latter in soft ground in the south approach.

All trenches were dug to exact depth, as no refilling below grade was allowed; then the excavation was made

for the bell end of each pipe, which was always the upgrade end. All joints were firmly made, then a gasket of hemp (of equal length to the circumference of pipe) dipped in a thick grout of neat cement, was compactly rammed into each joint with a yarning tool, care being taken that the joint at the invert remained even and smooth. Cement mortar was then firmly pressed into the joint with a wooden tool and finished off with a pointing trowel. Upon the completion of each joint, the pipe was cleaned out with a swab.

The amount of material in the sewerage system is as follows: 1200 ft. of 15-in. pipe; 216 ft. of 12-in. pipe; 400 ft. of 10-in. pipe; 4050 ft. of 8-in. pipe; 4030 ft. of 6-in. pipe; 36 manholes; 11 lamp holes; 5 flush tanks; 2 steel trestles.

Manholes are circular in form, with 8½-in. brick walls on a concrete foundation. The manholes used for flush tanks are very similar in design to the regular manholes except that they are not quite so deep and have in the bottom a cast-iron siphon.

Water for flushing these siphons is secured from the water mains and allowed to drip, so that the tanks flush about twice a week. This keeps the sewer clean, where at stub ends and at long runs having only a small amount of sewage, it might stop up.

Sewage-Disposal Plant—The sewers lead into a 15-in. pipe which runs on a grade of 2½% to the treatment plant on the shore of the lake. This plant comprises reduction chambers, anærobic filters and sprinkling filters, with a drop of 20 ft. from the receiving chamber to the effluent discharge. The plant has a capacity of 200,000 gal. per day.

The sewage enters a receiving basin or grit chamber 9x14 ft. and 10 ft. deep. From this it passes to two covered reduction tanks, 60x12½ ft., of 57,000 gal. capacity, its flow being retarded by baffles so that under normal conditions it remains about seven hours in the tank. It then flows over a weir into a rectangular well, from the bottom of which it passes through the side wall and flows across the bottom of the anærobic filter through lines of 4-in. tile laid with open joints. Each of the two filters is 32½x12½ ft., filled to a depth of 8 ft. with red granite, graded in size from 2 ft. of 6-in. stone at the bottom to 6 in. of ¼-in. stone at the top. A collecting trough at one end receives the overflow of the water issuing through the bed of stone and leads it to a siphon chamber.

From this chamber it is delivered intermittently by 6-in. siphons to a sprinkling filter. This has walls of perforated concrete blocks to admit air freely, loose stone being banked up against the inside of the walls to retain the main interior filling of crushed granite. The sewage from the siphons is delivered to rows of pipe fitted with spray heads of nozzles about 15 in. above the top of the filter bed, each nozzle being fitted with a deflector or splash plate. The effluent from this filter flows to a discharge pipe which extends about 150 ft. into the lake.

Storm-Water Drainage

The station is divided into four level plateaus by a small brook called Pettibone Creek, which enters at the extreme northwest corner and winds its way through the grounds to the inner basin at the extreme east edge. The drains empty into this creek at the most convenient locations, with concrete outlets to prevent wash by freshets. The tile and manhole work is similar to that of the sewer systems except in minor details. The drainage system comprises the following:

1,380 ft. of 18-in. mains	99 catchbasins
3,000 ft. of 15-in. mains	2 special catchbasins
1,040 ft. of 12-in. mains	41 manholes
3,160 ft. of 10-in. mains	6 lampholes
5,460 ft. of 8-in. mains	4 concrete outlets
10,255 ft. of 6-in. mains	2 concrete gutters in filled ground
100 ft. of 4-in. mains	

One of the 18-in. drains also drains the 125,000 gal. swimming pool in the basement of the instruction building. The hospital plateau has a separate system of drains, draining all surface and down spouts into the inner basin. Connections are made to the manholes, catchbasins and down spouts as on the main and receiving plateaus. The proposed contagious hospital has a separate 6-in. drain into the ravine directly east of the medical officer's quarters on the road around the inner basin, each catchbasin has a separate outlet into the inner harbor. The road leading to the main concrete arch bridge also empties independently into the creek.

Suggestions for Public-Utility Rate Making*

By W. G. Raymond†

Any public-service plant owned by a municipality (except possibly street-railways which are rarely so owned in this country) gives a certain service for which the whole taxable public should pay. A lighting plant lights the streets and a water-works system furnishes water for fires, for street cleaning, sewer flushing, and other public uses. It seems to be proper that all property served or to be served by water mains or gas mains which have been laid, or electric-light wires which have been strung and street lights operated, should pay a portion of the return that must be collected to cover the three items of interest, cost of operation and depreciation. The part that such properties should pay should be levied as a separate item in the tax, collected by the tax collector or treasurer and placed to the credit of the public-service plant for which the tax is levied. Thus there are three parts into which the income should be divided for collection. (1) A portion levied on every taxpayer for the general service to all citizens. (2) A portion levied on all property enjoying potential or actual service because of the existence of the service lines in its vicinity. (3) A portion paid by the users for the actual service rendered to them exclusively.

A municipally owned plant should pay the items of (1) interest; (2) operating expense, and (3) depreciation from the start. Since the number of customers and the quantity of output sold in the earlier years of the plant will be relatively small, it is suggested that in the beginning a relatively larger portion must be borne by the community as a whole for actual public use and potential private service, than will be proper in later years when a much larger output may be sold to individuals, and the common tax reduced.

The following is suggested as one way to apportion the amounts to be paid by the public at large and the individual users. A water-works is taken specifically and some modification of terms is needed in transferring the

*Extracts from a paper read at a "Municipal Lighting Day" convention, State University of Iowa, Iowa City.

†Dean, College of Applied Science, State University of Iowa.

plan to other utilities. Divide the loss of leakage between the users as a whole and the community as a whole in proportion to the quantities measured and estimated (for fires and streets) and augment the total private and the total public use thereby. A more or less arbitrary portion of the whole expense may be levied against real property served or to be served by the plant. What remains should be divided between the public at large and individual consumers in proportion to the total quantity of output used. So much as is to be paid by consumers for use should be arranged with a sliding scale, so that the larger users shall pay a lower rate, since with most public utilities the cost of service to a single user is not in proportion to the quantity of the output used, but is less per unit as the number of units is greater.

For water-works in small cities, the portion to be paid by the general public for fire protection is larger than would be obtained by considering only the amount of water used. Mains must be larger than would be required for ordinary business and domestic consumption, in order to furnish large quantities of water in short periods for fires. For the same reason, pumps must be of greater capacity than necessary for the domestic and business service. These differences may well be estimated and the interest and depreciation of the portion that may be necessary for fire protection only, should be included in the general levy on all taxable property, only the balance of these items being used in the sum to be divided as before described.

Let us suppose that the municipality prefers not to engage in the business itself but wishes to hire some person or corporation to do its business for it. Should there be any difference in the allowed rates? I think there should to the extent that the individual or corporation must be paid a wage for doing the business. This wage ordinarily will take the form of a larger return on the capital invested than mere savings-bank interest and should be such as to be fairly called interest and profit on the invested capital.

The business man in any private commercial enterprise expects to earn interest on his invested capital, wages for his time and energy in directing the business, and an extra profit as well. It may be said that he ought to expect to earn only interest and wage; this may be true, but then scarcely anyone will deny that the wage should be commensurate with the risk and magnitude of the enterprise. No one would expect a Marshall Field managing a business of millions of dollars yearly to be satisfied with the same wage that should satisfy John Smith who conducts a suburban corner grocery doing an annual business running not far into the thousands of dollars. And no man or group of men can be expected to undertake public business, unless a wage is permitted that is commensurate with the risk and the magnitude of the undertaking. Of course, the capital invested will largely measure the magnitude of the undertaking and yet this is not always so. A bank with a comparatively small capital may do a very large business and consequently earn a very large dividend on its capital, though not earning an exorbitant percentage of profit on the business done. Just so, by fortunate location and wise planning, one public enterprise may do a large business on a relatively small capital when compared with another less favorably located or less wisely constructed. Hence, it seems to me that if possible the profit or wage allowed the corporation should be allowed as a percentage of the cost of operation, it being understood that there will be supervision to the extent of preventing undue padding of this cost, and an agreement whereby the profit or wage may be larger as the cost per unit is smaller.

It would be well to make the articles of employment, called the franchise, such as to reduce the risk to the individual or corporation to a minimum. It might even be agreed that in return for good service with a plant reasonably well designed, constructed and operated, to the satisfaction of a municipality's engineering officer, a company should be guaranteed against loss from the start by rates levied much the same as for a municipally owned plant. Such a guarantee would lessen the risk and reduce the interest rate on the company's securities, and hence the total rate that the public should pay. And it might be further agreed that when the company should be on a paying basis, it should receive a higher wage in proportion as it increases efficiency and reduces the cost of service.

Thus a gas company might be allowed a standard rate of $1 a thousand cubic feet for gas and limited to an 8% dividend on its capital stock with a proviso that for each cent of reduction in the rate it would be allowed a small definite increase in its dividend. Such a practice obtains in London and in Boston though the figures used are not those cited above. Of course, this implies a public control of capital.

The one item that is really difficult to determine is "replacement" or, as it is generally called, "depreciation." There are many ideas as to how this item should be treated. It has been suggested that the sum to be laid aside annually for depreciation or renewals is that sum which placed at compound interest would produce the cost of the plant at the end of its life. But this may be questioned in the case of a municipally owned plant and it is certainly not the proper sum in the case of a privately owned plant. Renewals of parts of the plant will be required in a few years and will continue at intervals during the entire life of the plant, some items being renewed several times during the lives of other items. Therefore, it is not feasible to tie up the depreciation allowance at interest, unless, possibly, the lives of each class of items, short-lived and long-lived items, have been considered separately and depreciation allowances aggregated accordingly. In the case of a privately owned company, it is not proper to require the company to place the depreciation allowances at interest in order to realize the full investment by the time the plant has been entirely replaced. Rather should the cost of each part be returned to the investor by the time its life is ended. We should regard the depreciation allowance in the rates as a direct return of capital and should so arrange that it will be less at first and more at the last, since renewals are likely to be greater as the plant grows older. But it is also desirable for rate making that the total allowance for interest and renewals shall be practically uniform from year to year. The committee appointed by the American Society of Civil Engineers to report on methods and principles to be used in the valuation of public utilities has devised a way of meeting these requirements.*

By such a plan, the total depreciation payments made

*Discussed in "Engineering News," Feb. 12, 1914.—Ed.

during the life of the plant will equal the whole original investment, and the subtractions from capital will reduce the original capital to nothing at the end of the life of the property. If replacements have been prompt and the plant has been maintained at 100% effectiveness, it now represents just the capital originally put into it, and the payments from the public should go on as before. If it were possible to conceive of such a thing as the gradual and finally complete destruction of the plant by reason of its use and the failure of the company to reinvest in it the sums allowed for depreciation, and this result should obtain at the estimated end of the life of the property, there would be no further service, and no further payments due. The company would have received interest on all the capital actually invested in the plant at all times up to the end of its life, cost of operation, profit or wage for its service, and also the capital it had originally invested. It would thus stand even, having received interest on its capital while in use and pay for the service given. Of course, no such thing actually could happen because if the plant were allowed to run down, the service could not be maintained and would cease to be efficient long before the end of the life of the plant. The property must be considered to be a permanently continuing enterprise.

If the same rate of interest is used for return on capital and for depreciation allowances the aggregate annual payments will be equal. If the interest allowed on capital differs from that allowed in computing depreciation the payments will not be quite uniform and if the interest on capital is the greater, the aggregates will be somewhat greater at the first than at the last. This is undesirable but cannot be avoided under the plan which allows profit in the shape of large interest on invested capital. I should like to suggest that the interest be the same on capital and depreciation allowances and that profit or wage be computed on cost of operation.

No attempt has been made to discuss the subject of valuation of a public-utility property, effort being confined to showing how rates may be established when the value is known.

New Canal and Locks at "The Soo"

SYNOPSIS—The locks at Sault Ste. Marie, in the channel connecting Lake Superior with the lower lakes, form a feature in the inland navigation system of the Great Lakes, for at this point passes nearly 75% of all the lake traffic, while the traffic through the locks in 1912 amounted to over 72 million tons of freight. To increase the facilities for handling the traffic on the American side, a lock of exceptional size is under construction, as an addition to the two existing locks, while excavation has been commenced for a fourth lock. This constitutes a very important engineering work, both in design and construction, and is described in the accompanying article.

The canals and locks which enable lake vessels to pass the rapids in the St. Marys River at Sault Ste. Marie are among the most striking features of the engineering work in connection with navigation on the Great Lakes. The St. Marys River, forming the connection between Lakes Superior and Huron, has a length of about 64 miles, including 35 miles of improved channel. As now improved, its navigable portions have channels 300 to 1000 ft. wide, with a minimum depth of 21 ft. at low water. At Sault Ste. Marie, the river has a stretch of rapids about three-quarters of a mile in length, with a fall of 17 to 21 ft., according to the relative stage of water in Lakes Superior and Huron. The width of the river here is about half a mile. Navigation is provided for by a canal with two parallel single-lift locks on the American side and a canal with one single-lift lock on the Canadian side.

The traffic to be handled is enormous, and navigation is stopped by ice for about four months each year (ordinarily from Dec. 15 to Apr. 15). Consequently, it is of the utmost importance to handle the vessels promptly and enable them to make as many trips as possible during the navigation season. It became apparent some years ago that the capacity of existing locks to handle vessels promptly and economically would soon be reached.

In 1905, Congress authorized the widening of the canal, while in 1907, an appropriation was made for the construction of a third lock with an independent canal. These improvements were described in ENGINEERING NEWS of Sept. 7, 1911. The canal widening was completed in 1911, and the new lock and canal are expected to be in operation at the opening of navigation in 1915, or possibly earlier.

After these improvements had been authorized, it was considered desirable to duplicate the new lock, not so much to increase the traffic capacity as to provide against interruption to traffic in the event of any accident at the third lock. This duplication is a feature of the locks on the Panama Canal, where the traffic will be much less in volume than at "the Soo." The additional lock was authorized by Congress in 1912, and work is now in progress on the excavation. The time of its completion depends upon the appropriations made by Congress. The general situation on the American side is shown by the plan, Fig. 1.

With all this work completed there will be the unique spectacle of two parallel and adjacent canals with four parallel single-lift locks. The dimensions of the locks are as follows:

Lock	Year opened	Size of chamber	Water on sill*
1. Weitzel	1881	515x80 ft.	12.6 to 17 ft.
2. Poe	1896	800x100 ft.	17.7 to 22 ft.
3. Third, est.	1915	1350x80 ft.	24.5 to 25 ft.
4. Fourth, est.....		1350x80 ft.	.....

*The first figure in each case refers to the established low-water plane.

It will be noted that the width of the two new locks (80 ft.) is less than that of the second lock (100 ft.), and the same as the first lock. The greater width of the second (Poe) lock was adopted to increase its capacity by enabling ships to lie side by side. In operation, however, it is found that this arrangement causes delay in getting the vessels into position and then working them out through the gates. It will not be followed, therefore, in the new locks. These latter are made long enough to take two of the largest vessels end to end, so

that they can enter and leave without any maneuvering in the lock chamber. But while the width of the chamber is the same as that of the first lock, this width is carried through the gates or entrances, instead of being reduced to 60 ft. at the entrances, as in the first lock. The largest freight vessels on the lakes at present have a beam of about 64 ft. There are a few passenger vessels with beam greater than 80 ft., which occasionally pass "the Soo," but these can use the 100-ft. Poe lock.

General Review

The first canal at "the Soo" was built on the Canadian side by the Northwest Fur Co., in 1797-98, and had a boat lock 38x8¾ ft., with a lift of 9 ft. In 1853-55, the first canal on the American side was built by the State of Michigan, a right-of-way through the Government reservation having been granted by the United States, and a grant of 750,000 acres of land made by Congress to assist in the construction. The canal was 1.8 miles long, 13 ft. deep, 64 ft. wide at the bottom and 100 ft. at the water level. It had two tandem locks 350x70 ft., with 11½ ft. of water on the sills and a lift of 9 ft. each. These were destroyed to make room for the present Poe lock.

The Weitzel lock, the first of the existing locks, was built by the U. S. Government during the years from 1870 to 1881. It is 515x80 ft., but the width is reduced to 60 ft. at the gates. During its construction the canal was enlarged to a width of 160 ft. and a depth of 16 ft., with vertical sides formed by rock-filled cribs resting on the solid rock. The canal was turned over to the Federal government in 1881. The Poe lock was built by the U. S. Government in 1887-96. Since 1892, the canal has been deepened to 25 ft., and widened to 400 ft. at the entrance, the width reducing to 300 ft. where it is divided by a pier between the two lock entrances. At one point, however, the width was reduced to 108 ft. at a railway drawbridge, which caused considerable delay as vessels could not pass here. At this point, the widening of the canal (mentioned above) formed a second 108-ft. channel with the pivot pier of the railway swing-bridge and the movable dam on an island, so that vessels can move in both directions without interference.

The present canal and lock on the Canadian side were built by the Canadian Government, 1888-95. The canal is 1⅛ miles long, 150 ft. wide and 22 ft. deep, with a lock 900x60 ft., having 18.2 ft. of water on the sills at established low water. No tolls are charged on either the American or Canadian canals, so that vessels use whatever is most convenient. As the Canadian lock affords a slightly greater draft for down-bound vessels than does the Poe lock, it is customary for deeply loaded down-bound boats to take the Canadian lock, while the greater number of up-bound boats take the Poe or Weitzel locks.

Hydraulic power is used for operating the gates, etc., at the American locks, under a pressure of 115 lb. for the Weitzel and 200 lb. for the Poe lock. Electricity generated by a water-power plant at the canal is used on the Canadian side, and the new locks on the Ameri- and Poe locks can each be emptied or filled in about 8 min., and the gates can be opened or closed in 2 min., while both these operations at the Canadian lock take about 8 min. The average time for a lockage, however, is 33 min, at either of the American locks and 19½ min. at the Canadian lock, the time being dependent largely on the number of boats in the lock at one time. The cost of the several improvements up to the present time is approximately as follows:

Locks and canal of 1855	$1,000,000
Weitzel lock	1,000,000
Poe lock	3,000,000
Improving canal	4,400,000
Improving river channels	9,400,000
Canadian canal and lock	5,000,000
Total	$23,800,000
Third lock (estimated)	$3,200,000
Second canal (estimated)	3,000,000
Fourth lock and approaches (estimated)	3,275,000

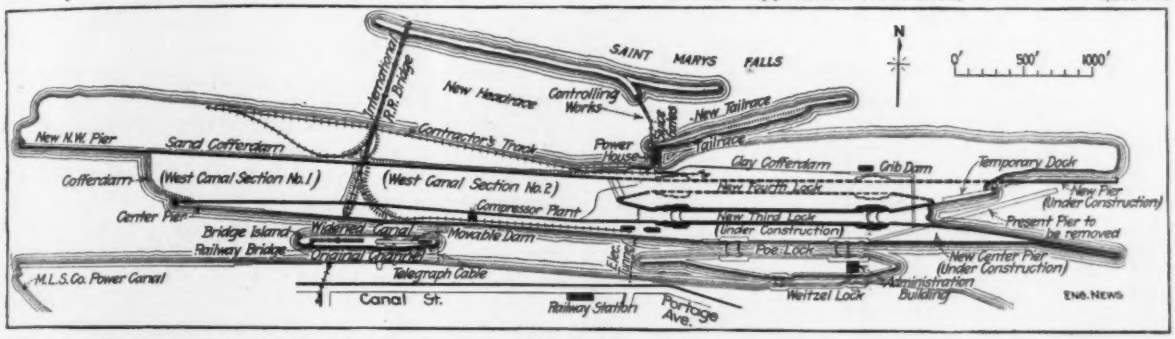

Fig. 1. Plan of Canals and Locks at Sault Ste. Marie, Mich.; Showing the Second Canal and the Third and Fourth Locks Now under Construction

The annual cost of operation and maintenance of the American locks has increased steadily from $27,546 in 1882 to $152,197 in 1912. But this cost expressed as the rate per ton of freight passing the canal has declined in the same period from 13.57 mills to 4.64 mills (with a maximum of 16.03 mills in 1883 and a minimum of 1.98 mills in 1906). The operating force numbers 77, working in three 8-hr. shifts; and there are also about 20 laborers, clerks, watchmen, etc. The offices are in a stone building between the two existing locks.

The canal and locks are under the jurisdiction of the U. S. Corps of Engineers (War Department), and the engineer officers in charge of the district which includes this canal have been as follows: Capt. A. W. Whipple, 1848-61; Col. T. J. Cram, 1866-70; Major O. M. Poe, 1870-73; Col. G. Weitzel, 1873-82; Major F. U. Farquhar, 1882-83; Col. O. M. Poe, 1883-95; Col. G. J. Lydecker, 1896-1902; Major W. H. Bixby, 1902-04; Col. C. E. L. B. Davis, 1904-08, and Col. C. McD. Townsend, 1908-12. The present officer in charge is Lieut.-Col. Mason M. Patrick.

The operation and maintenance work, and also the construction work on the new locks, are under the direct

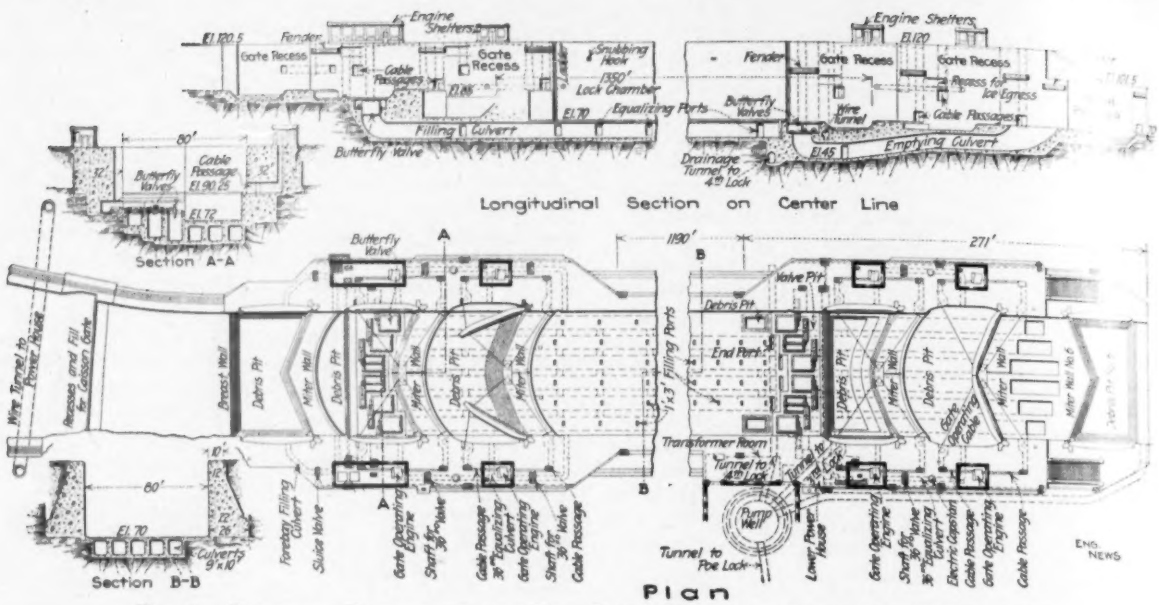

FIG. 2. PLAN AND SECTIONS OF THE NEW LOCK (NO. 3) AT SAULT STE. MARIE, MICH.

charge of Louis C. Sabin, General Superintendent. For the new work, W. J. Graves and I. De Young, Assistant Engineers, have done much of the designing and are in immediate charge of the construction.

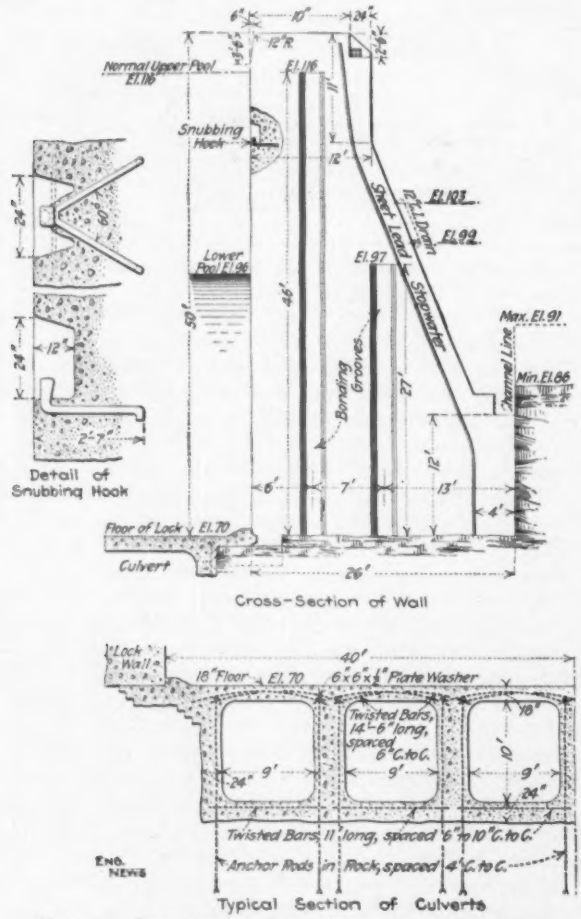
Normal Upper Pool El. 116
Snubbing Hook
Lower Pool El. 86
Detail of Snubbing Hook
Sheet Lead & Stopwater
Bonding Grooves
Max. El. 91
Normal Line
Min. El. 86
Floor of Lock El. 70
Culvert
Cross-Section of Wall
Lock Wall
18" Floor El. 70
6"x6"x1/2" Plate Washer
Twisted Bars, 14'-6" long, spaced 6" C. to C.
Twisted Bars 11' long, spaced 6" to 10" C. to C.
Anchor Rods in Rock, spaced 4' C. to C.
ENG. NEWS
Typical Section of Culverts

FIG. 3. SECTION OF LOCK WALL AND CULVERTS

PRESENT IMPROVEMENTS

The improvement work now in hand includes the following: (1) building the west canal to the new locks; (2) building lock No. 3; (3) building the lower approach to the new lock; (4) excavating for lock No. 4. The present article deals particularly with the new lock No. 3, work on which is well advanced.

The contractors are as follows: John Marsch, Chicago, completed the excavation for the third lock; the MacArthur Bros. Co., Chicago, has the contract for the new upper canal west of the locks and for the excavation of the lock pit for lock No. 4; the Great Lakes Dredge & Dock Co., Chicago, has the contracts for the concrete work of Lock No. 3 and the construction of the lower entrance to the two locks.

THE NEW LOCK, NO. 3

This lock, as noted above, will be 1350 ft. long inside the gates and 80 ft. wide. The walls of the lock chamber are 50 ft. high above the floor, with a thickness of 12 ft. for about 11 ft. from the top; below this point the back has a batter of 2½ on 1 to the footing, which is 26 ft. wide for a height of 12 ft. above the base. At the back, near the top, is a shelf for the electrical conduits. The floor was built first, its outer edge being formed with a projecting rib or tenon to bond into the toe of the wall. Fig. 2 shows the general design of the lock and its appurtenances, and Fig. 3 gives the details of the lock wall and culverts.

The walls were built in alternate sections 30 ft. long and in each end of each section were formed two vertical bonding grooves 2 ft. wide and 8 in. deep. An expansion joint is formed by facing the ends of the form with gravel-coated roofing felt, left in place, and in order to seal this joint against leakage of water from the rear, a lead stop-water is formed (Fig. 3). In each end of each alternate section, at 2 ft. from the rear face, a 14-in. sheet of lead is embedded 6 in. in the concrete and crimped to form a rib 1 in. high in the joint. This rib lies against the end of the concrete section, filling the

expansion joint space, while the remaining 6 in. of the width projects so that it will be embedded in the adjacent section.

To intercept stones, water-logged wood, etc., carried in with the water from the canal, there are three débris pits in the floor of the upper lock entrance, one at each set of gates. Three pair of débris pits are provided in the floor to intercept similar material which may get into the lock chamber and roll along the floor.

Electric capstans will be provided for operating the gates in case of breakage of the gate machinery. They may also be used in placing vessels in the locks, but this is seldom necessary, as lake vessels are usually provided with steam winches for warping. Cast-iron mooring posts will be provided along the top of the wall. In the face of each wall of the lock chamber snubbing posts are placed in recesses 24x18 in., 12 in. deep, 60 ft. apart, and 10 ft. below the top of the wall. The hook is of wrought iron, with a vertical head in the recess and two legs forming a V embedded in the wall.

Culverts and Valves—The filling of the lock is effected by means of culverts beneath the floor, having numerous port openings through the floor. Above the upper gates is a valve pit with vertical openings into six filling culverts, each opening being fitted with a butterfly valve. Equalizing ports are provided in the partition walls between the culverts to relieve the horizontal stress on the partitions and to better distribute the flow. The filling ports are 1x3 ft., spaced about 30 ft. apart at the upper end and 15 ft. at the lower end of the lock chamber. The two culverts next to the side walls extend only about half the length of the lock, and each has a manhole port 2x9 ft. at the end. The four central culverts extend the full length, and end at similar ports.

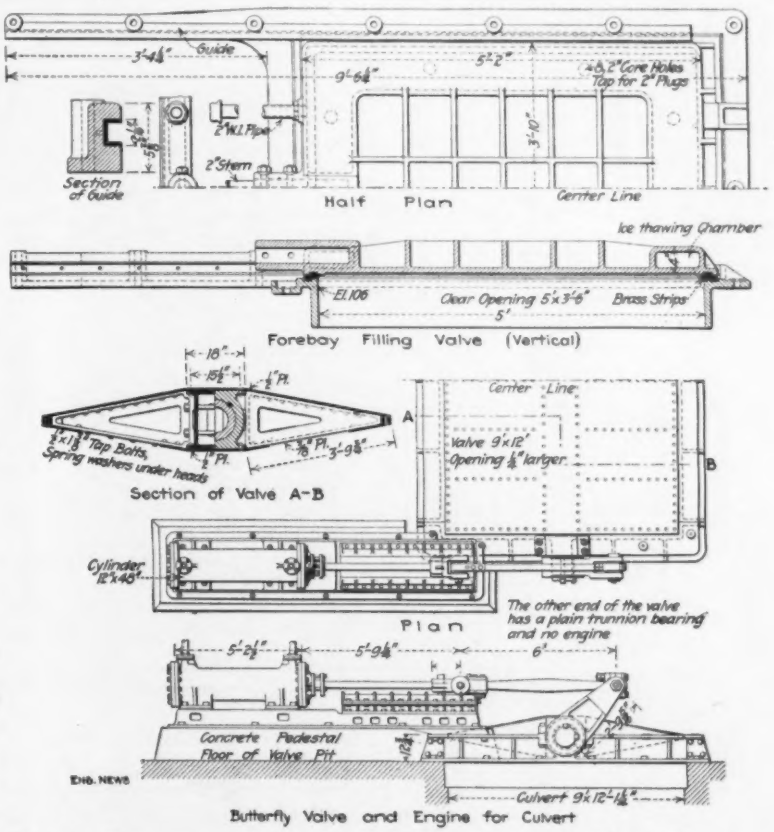

Fig. 4. Butterfly Valves for Lock Culverts and Sluice Valves for the Forebay

For emptying the lock, there are six discharge culverts, opening from a valve pit immediately above the first set of lower gates, and discharging through enlarged openings in the floor of the lower entrance, above the guard gates.

The filling culverts are all 9 ft. wide and 10 ft. high, built of reinforced concrete, with specially heavy reinforcing in the 18-in. roof (forming the floor of the lock). To anchor the culvert tops against upward pressure, anchor rods from 1 to 1½ in. diameter are set 4 to 7 ft. deep in the rock, passing up through the culvert walls and having at their upper ends washer plates and nuts beneath which pass the longitudinal bars which hold the transverse rods of the roof reinforcement. The anchor rods of each row are spaced about 4 ft. apart, longitudinally, those of adjacent rows being staggered. All the culverts are built upon the rock.

The reinforcement in the 24-in. bottom of the culvert consists of ⅜-in. transverse bars 11 ft. long, 6 to 10 in. apart. The reinforcement in the 18-in. roof of the culverts consists of ⅞- to ¾-in. transverse bars, 14 ft. 6 in. long, 6 in. apart. These are curved, as shown, and their ends are held down by a pair of ⅞-in. longitudinal bars upon which rest the washer plates of the vertical anchor rods. In the lower part of the chamber, where there are only four culverts and the upward pressure is less, the reinforcement is slightly reduced. Here, also, a strip of floor 24 in. thick extends from the culverts to the side walls. This has a width of 13 ft. 6 in., with a longitudinal expansion joint at its junction with the culvert roof and a dovetail joint bonding it to the side wall.

At the upper entrance of each culvert is a butterfly valve, 9x12 ft., revolving on trunnions and being horizontal in its closed position. The valve is hollow and approximately oval in section, built up of steel plates and castings. Each one is operated by a horizontal hydraulic piston-engine, 12x48 in., placed in the valve pit. The piston-rod cross-head is connected by a 6-ft. connecting-rod to a 34-in. crank arm on one of the valve trunnions. These valve engines work under 200-lb. pressure. The butterfly valves are shown in Fig. 4.

As both sets of gates will be used at each lockage, equalizing culverts in the side walls provide for regulating the height of the water inclosed between the gates. At the upper end there is a forebay filling culvert forming a bypass around the guard gates, while a second equalizing culvert connects this forebay with the lock

chamber and has a branch to the space between the two sets of service gates. The former bypass is controlled by a sluice valve 48x60 in. and the latter by two 36-in. gate valves, one above and the other below the lateral branch. A similar equalizing culvert, with a 36-in. valve, connects the lock chamber with the space between the lower gates, but there is no bypass around the outer pair of these gates or around the lower guard gates.

The two forebay filling valves (Fig. 4) are flat cast-iron sluice gates, sliding vertically and guided by side rods moving in sleeve guides; each is operated by a threaded stem in a nut driven by gearing. They give clear openings 3½x5 ft. To thaw the valve when frozen fast, a thawing chamber is cast upon the back, all around the edge. The 2-in. guide pipes open into this chamber, and provide for steam circulation, hose pipes being coupled on when necessary.

The service gates are of steel, the main portion consisting of horizontal plate-girders and vertical frames or posts, with skin plating on the upstream side. The gate construction is shown in Fig. 5. The girders are 4 ft. 3 in. wide, with a spacing of 2 ft. 8 in. in the lower portion and increasing to 3 ft. 4 in. near the top. Each girder has four flange angles and a web plate, with a 10-in. cover-plate on the upstream chord, while the ends of the downstream chords are riveted to vertical plates ¾x45 in., extending the full height of the gate. For diagonal bracing, to carry the overhanging load, there is a 14-in. plate or strap extending from the top hinge to the bottom outer corner, with two other 10-in. diagonals. These are on the downstream side and are riveted to the chords of the horizontal girders at each intersection.

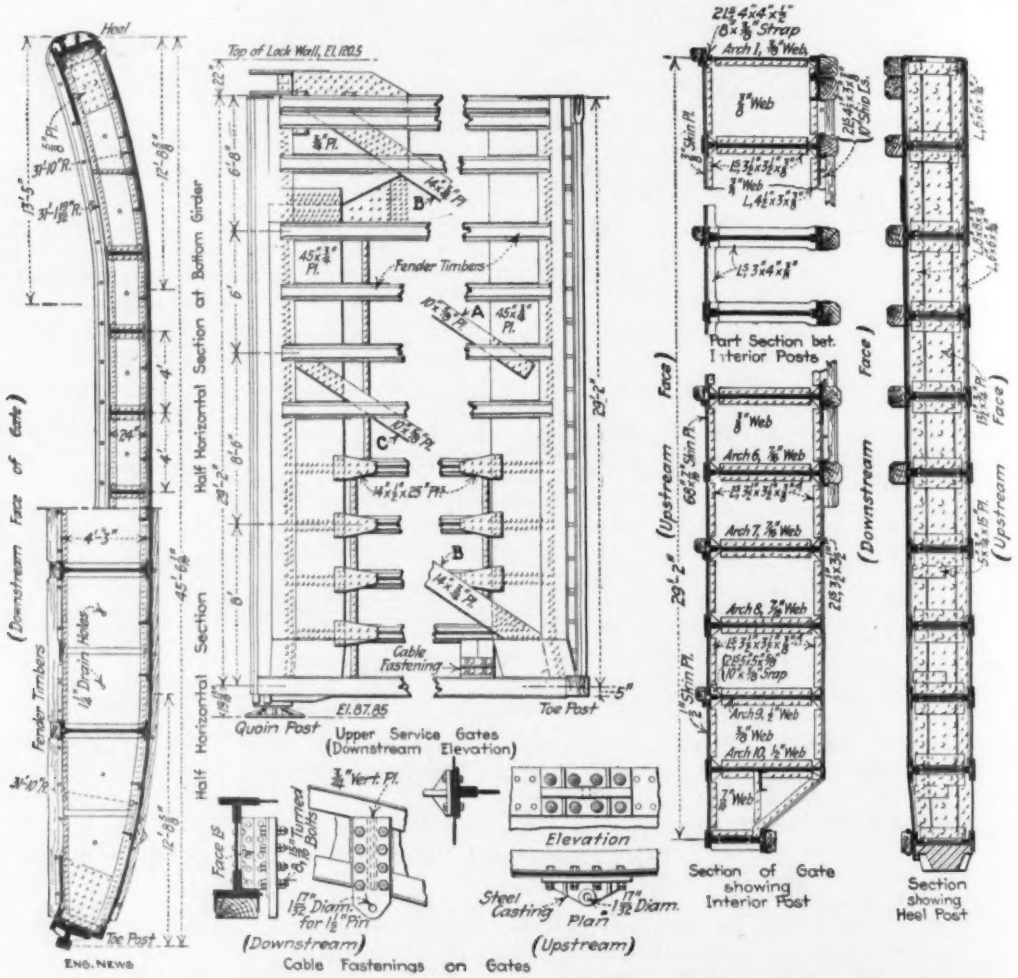

FIG. 5. STEEL MITERING GATES FOR THE NEW LOCK AT SAULTE STE. MARIE

The bottom girder is only 2 ft. wide, and its downstream chord has a heavy cover-plate and a 10-in. channel to receive the oak sill or closing timber. This narrow bottom girder is to permit the gate to lap over the sill, thus reducing the area subject to uplift when the gate is closed.

Each gate leaf is 45 ft. 6 1/16 in. long over the posts. The lower gates are 51 ft. high from center of sill cushion to foot-bridge. The upper gates are 31 ft. high. The angle between the leaves when closed is 136° 24'. The

LOCK GATES

Each end of the lock will be equipped with two pair of steel service gates and a pair of timber guard gates, all being of the mitering type. No emergency dam will be placed in the upper canal (as in the present canal), but safety will be insured by using both sets of service gates for every lockage.

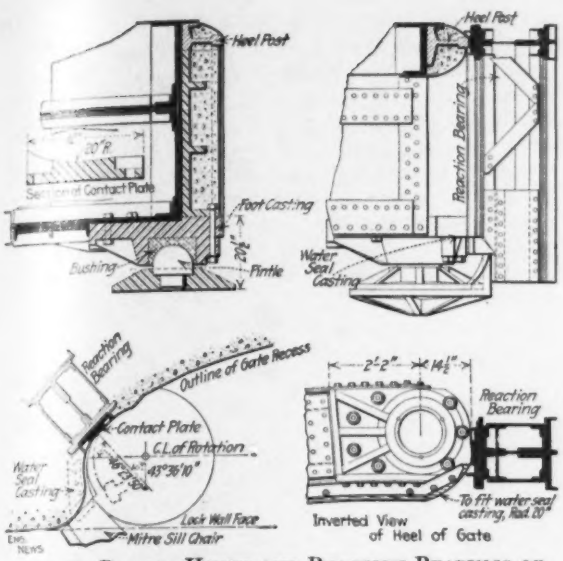

FIG. 6. BOTTOM HINGE AND REACTION BEARINGS OF LOCK GATES

rise of sill, or distance from the line through center line of quoin contacts to the similar line of the toe contacts when the gates are closed, is 16 ft. 10⅛ in. The gates are not curved in plan, but the downstream side of each leaf is straight, while the upstream side is straight for the center portion and then curves in toward the heel (or quoin) and toe posts.

These posts are built up of steel castings of T-section, the broad base being bolted with finished bolts to the gate frames and the narrower head forming the contact piece. Curved plates are fitted on each side of the castings at the heel, and the space within these is filled with concrete. The contact face of the toe post is fitted with a wearing piece of channel section.

The upper hinge has an 8-in. pin fitted to pin plates on the gate and to turnbuckles on the anchorage. The bottom hinge consists of a forged-steel hemispherical-headed pintle of 12-in. diameter in a steel casting resting on a base-plate which is bolted to the reaction bearing. This pintle engages with a phosphor-bronze bushing in a casting on the gate post, as shown in Fig. 6. When the gates are closed, semicircular recesses in castings on top of the toe posts form a cylindrical socket into which a locking pin or plug is dropped by the gateman.

When open, each leaf will be held firmly in the gate recess by a latch, so that in case of accidents to other gates permitting free flow of the water the gate will not be carried away by the current. These latches will be operated by cable from the operating houses. At the Poe lock, the latches on one pair of gates are thus operated, while the other gates have not as yet been so equipped but are held by hooks operated by hand. Recesses in the lock wall, at the end of each gate recess, facilitate poling out ice or drift that may collect behind the gate as it opens.

Heavy fender timbering is provided on the downstream side of the gate, and extends below the low-water line, to protect the gate when open and in the recess. The timbers are fitted to 10-in. steel ship channels. The vertical channels and timbers (7x9¼ in.) are continuous; the horizontal channels and timbers (8½x9¼ in.) being cut and framed between them. On the upstream side there are horizontal fender timbers 4⅛x5 in., carried by shelf angles. All these timbers are of oak.

The reaction bearings (Fig. 6) are heavy box-girders, built into the concrete of the hollow quoins. They are lined with steel castings bolted to the girders and having the face flush with the masonry forming the contact face. The radius of the contact faces of this casting and the heel casting on the gate is 20 in. The center of rotation of the gate is 5¾ in. from a line through the center line of the contact, this eccentricity relieving the bearing as soon as the gate starts to open.

The box-girder reaction bearing has two side webs and a central web-plate (at right angles to the others), with heavy latticing over angles riveted to the edges of the side webs. The foot of the column is riveted to a cast-iron base plate. The outer face of the column has a cover-plate 20x1 in., and after the column had been built up, a groove 12 in. wide was planed in this so as to produce a perfectly plane surface for the entire length, to receive the cast-steel reaction bearings. These are put in place before the column is in position, but may be adjusted later as they are secured to the face-plate by phosphor-bronze tap bolts set by cup wrenches. The castings are in lengths of 6 ft. 8 inches.

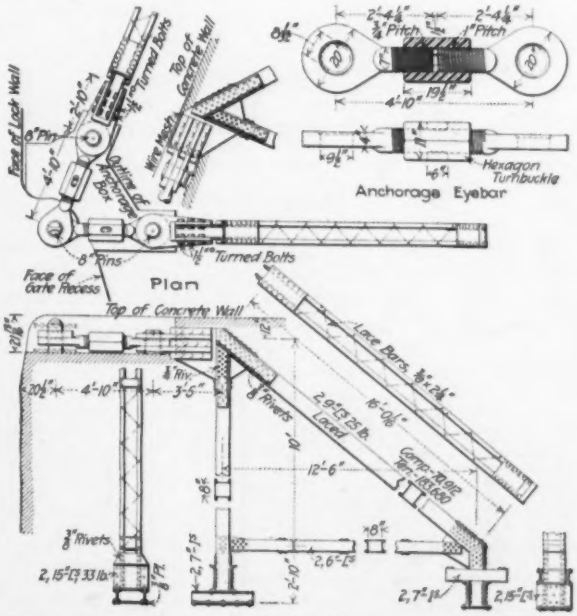

FIG. 7. ANCHORAGE FOR TOP HINGE OF LOCK GATES

The anchorage embedded in the concrete to hold the top hinge, instead of being composed of eye-bars (as at many locks) consists of built-up structural steel A-frames or trusses which are set approximately parallel with and at right angles to the lock wall, with their upper ends attached to pin plates for two 8-in. pins. This construction is shown in Fig. 7. Short anchor bars connect these pins with the 8-in. hinge pin of the gate, each anchor consisting of a pair of short eye-bars having threaded stems connected by a hexagonal turnbuckle so that the length is adjustable. The bars have heads 20 in. diameter and 4 in. thick, with 9½-in. holes to receive bronze bushings for the 8-in. pins. The threads are

square, of ¾-in. pitch on one bar and ½-in. on the other. the differential screw giving a movement of ¼ in. for each complete turn of the nut. For turning the turnbuckle a 6-ft. forged-steel wrench is provided.

At each end of the lock, beyond the main gates, will be a pair of timber guard gates (Fig. 8). Each leaf is composed of horizontal courses of yellow fir, giving a solid horizontal thickness of 31½ to 45½ in. The timbers are held together by 2-in. rods extending the full

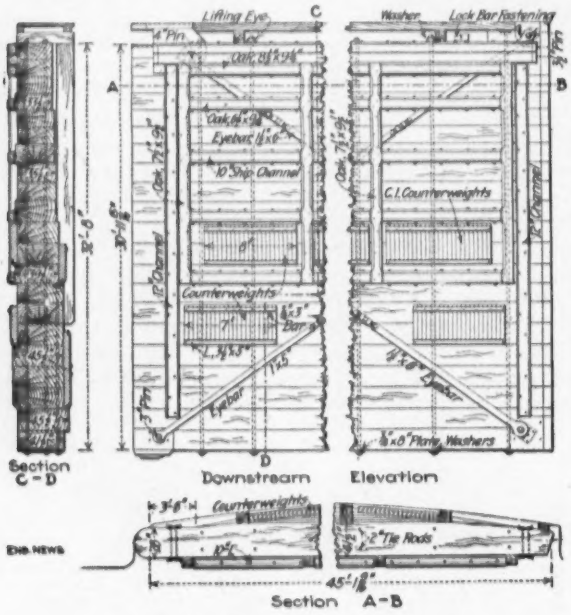

FIG. 8. TIMBER GUARD GATES

depth of the gate, and by two vertical 12-in. steel channels bolted together at each end. Diagonal bars, having eye-bar ends and set up by turnbuckles, extend across each leaf, their ends being fitted to 4-in. pins in screws. The height of the gates is 32 ft. 8 [illegible] for the lower gates and 31 ft. 2 in. for the upper gates.

GATE-OPERATING MACHINERY—The gates will be operated by steel cables and electric winding engines. This system was adopted on account of its simplicity, flexibility and ease of repair; and because it keeps all gate attachments in deep water and out of the way of ice. With the jack-arm system used in many locks there would be liability of getting the mechanism frozen up, delaying operation at a critical time when it is desired to handle the traffic quickly before navigation is closed by ice. A deep catch pit or débris pit in front of each sill prevents material from lodging and obstructing the gate.

Each gate leaf is operated by two ⅞-in. wire cables, one for opening and one for closing, arranged as shown

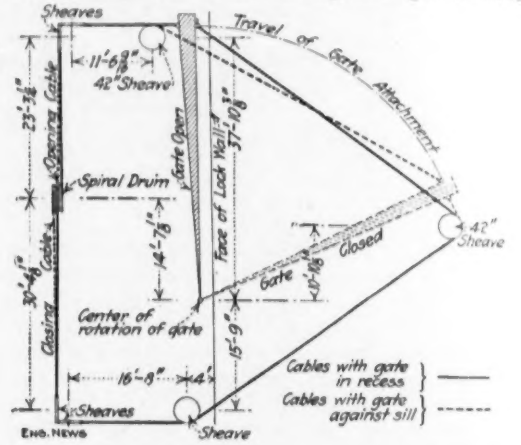

FIG. 9. PLAN SHOWING ARRANGEMENT OF CABLES AND ENGINE FOR OPERATING THE LOCK GATES

in Fig. 9. One end of each is attached to the operating drum, the attachment consisting of a hollow screw to which the cable is secured by clips (Fig. 10). A nut riding on the screw enables its position to be shifted, to

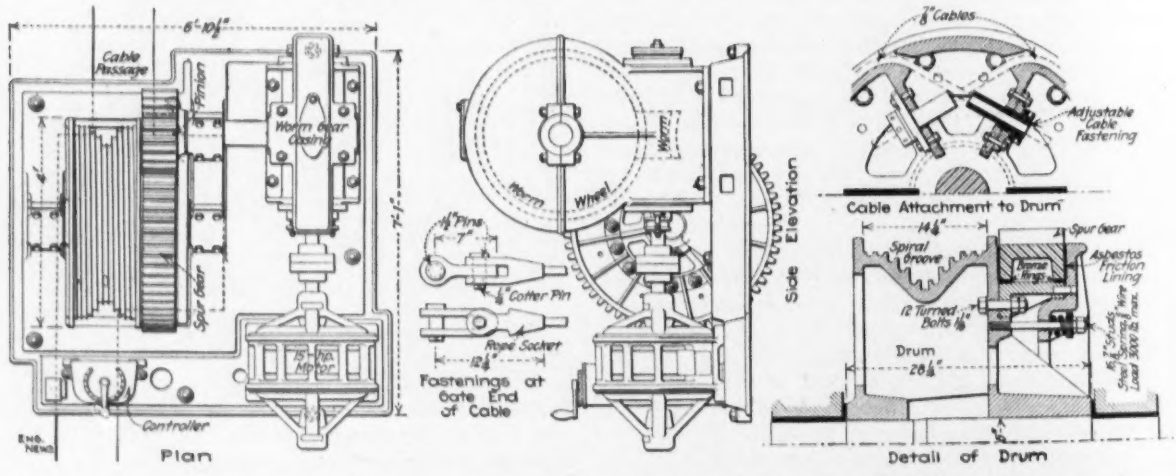

FIG. 10. ELECTRIC WINDING ENGINE FOR OPERATING THE GATE CABLES OF THE NEW LOCK AT SAULTE STE. MARIE, MICH.

steel pin plates. There is also oak fender framing on the downstream face. To overcome the buoyancy, cast-iron blocks are fastened to the face of the gate by steel bars and supported by shelf angles secured to the gate by lag

adjust the length of the cable. The other end of the cable is attached through a bell clevis to a pin and swivel on the gate (Figs. 5 and 10). The drum is 4 ft. diameter and has a double spiral groove specially laid out

to compensate for the variations in the length around the cable system as the gate swings through its arc, and thus maintain both cables taut at all times. The cables pass around 42-in. grooved sheaves for changes in direction, one of these sheaves being on the top of the miter wall or on the floor just downstream from the sill.

The electric winding engine (one for each gate leaf) is shown in Fig. 10. The drum is driven by gearing from a 15-hp. induction motor of the slip-ring type, running at 450 r.p.m. To the motor shaft is coupled a single-thread worm gear, 9¼-in. pitch diameter, with a ratio of 47 to 1, and this engages a worm wheel 39.34 in. diameter. On the shaft of this wheel is a pinion gearing with a spur wheel on a ring bolted to the face of the

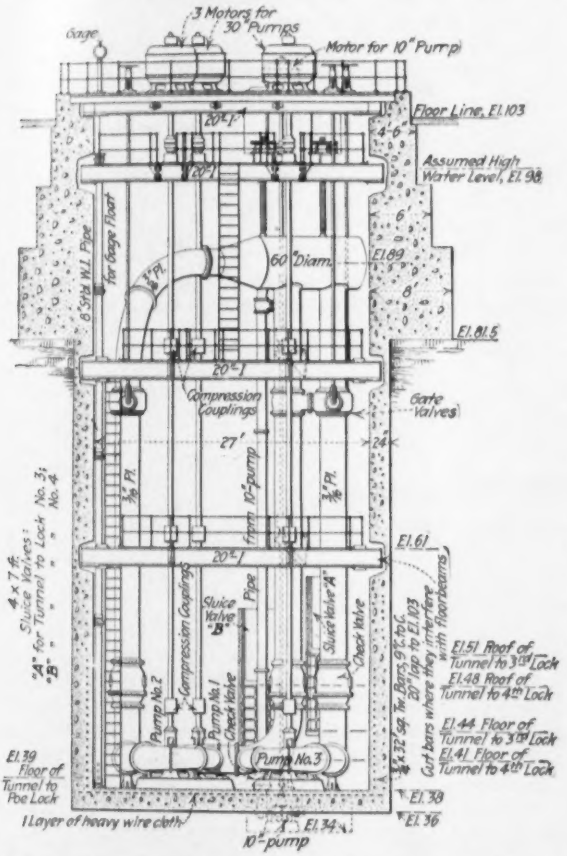

FIG. 11. PUMP WELL FOR SERVING THREE LOCKS AT SAULTE STE. MARIE, MICH.

cable drum. The gears are of cast steel (except that the worm wheel is of bronze), and all have cut teeth. A slip friction ring on the drum allows it to give and relieve the load in case of sudden shock. The motor, gears, drum and controller are alll secured to a single bedplate and the plant for each gate leaf will be housed in a separate concrete shelter on the lock wall.

The current necessary for the operation of the motors of the lock machinery will be supplied from a power plant owned by the government and operated by a private company, as described later.

PUMPING PLANT

The pump-well designed formerly for the third lock is provided with two tunnels leading toward the Poe and fourth locks, so that when completed, any one of the three locks may be pumped out with one plant. The pump well (Fig. 11) is placed between the Poe lock and Lock No. 3; the tunnel to No. 4 passes beneath Lock No. 3. The tunnels are 4x7 ft., with vertical sides, flat floor and arched roof. From the well there is a single steel discharge pipe of 5-ft. diameter, laid along the back of the lock wall and discharging through this wall below the guard gates. The pump well is 27 ft. diameter, and has 24-in. concrete lining, reinforced in the lower portion, while the concrete floor is reinforced with a layer of wire cloth.

At the bottom of the well are three 30-in. centrifugal pumps, with vertical shafts, each driven by a 300-hp. electric motor at the top of the shaft. Each pump has a discharge capacity of 27,700 gal. per min., at 385 r.p.m. For pumping ordinary leakage there is a 10-in. pump set in a sump in the floor, and driven in the same way by a 60-hp. motor at 870 r.p.m. The sluice valves in the well, at the mouths of the three tunnels, are operated by a threaded stem passing through a nut which is driven from a hand-operating stand.

(Note—The construction work on the new lock No. 3, and some other features of the extensive improvements at "the Soo," will be dealt with in a separate article.)

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The Chicago Drainage Canal and its auxiliary works are reviewed in the annual report, for 1913, of Mr. Smyth, President of the Sanitary District of Chicago. The main canal is being widened in the earth section, to give a flow capacity equal to that of the rock section. During 1913, a 70-ft. widening was made for five miles, leaving 1½ miles yet to be done, and an additional widening of 22 ft. for the entire distance will be required. Five sections of the Sag or Calumet channel were let at $2,036,114, and five miles of channel for draining the Sag Valley were built by force account. About four miles of the Sag channel are yet to be let. The North Shore channel, from the Chicago River to Wilmette, has given continual trouble, due to sliding and caving of the banks, and about 160,000 yd. of this material was removed at an expense of about $107,000. Experiments are being made with methods of stopping the slides. Two sections have been let for the North Shore intercepting sewer, extending from Wilmette to Glencoe. The purpose of this is to keep the sewage of the north-shore towns out of the lake, and it will cost about $375,000.

A removal of additional obstructions in the Chicago River is recommended for this year. In 1913, about 121,000 yd. of septic sludge were excavated from the Bubbly Creek arm of the river, in the stockyards district, and for the first time the various firms in this district have paid part of the expense (paying $26,000). About 80,000,000 gal. of water daily have been pumped into this branch to remedy conditions, but the problem of abating the nuisance is a difficult one. The sanitary district has had an experimental station at the stockyards for more than a year, investigating the matter of taking care of the trade wastes which cause the trouble.

About 80 sq.mi. of land have been added to the district by recent legislation, and this includes the territory along the Des Plaines River. The outfall sewer from Cicero to the main channel at 52nd Ave. is under construction and will cost about $150,000. An outfall sewer for the town of Berwyn is being constructed also, and some means of relief must be provided for Morton Grove and towns north of this in the Skokie valley.

It is expected that the question of the right of the sanitary district to take water from Lake Michigan to the full extent of the capacity of its channel will be settled soon in the federal courts. Mr. Smyth states that if the district is limited to 4167 cu.ft. per sec., instead of the desired 14,000 cu.ft. per sec., it will be necessary for the district (or the various municipalities within its limits) to build sewage-purification works at a cost (including operating expense) of about $200,000,000. He adds: "It is very doubtful, judging from past experience, that any such works would accomplish a result even approaching that which would be obtained by the dilution method now in use by the district."

Surface Drainage at Street Intersections

By Paul E. Kressly*

It is usually difficult to decide on the best form of crown for the pavement and on the method of surface drainage at street intersections, for there is no uniform practice among municipal engineers, and the actual practice varies so materially in different cities. There are, however, certain objects to be attained, which govern the design, all of which must be given consideration before arriving at the proper solution of the problem. The more important of these objects are: the good appearance of the street, the prompt removal of the surface water and the convenience and comfort of pedestrians and vehicle users.

Maximum Grades across Intersections—There is much diversity of opinion as to the best arrangement of grades at street intersections, and no hard and fast rule can be laid down to fit all conditions. For vehicles to pass the intersection smoothly the crown of each roadway should be continuous across the intersection. This condition is not always possible, especially where the intersecting street grades have much inclination.

At Inglewood, I have obtained satisfactory results by continuing street grades of 3% or less unbroken across the intersection. When grades are steeper than 3%, the best method is to break grade at the curb line and cross the intersecting street at a grade not exceeding 3%. The grade between property and curb lines should not be more than 6%. Such an arrangement of street grades gives satisfactory pavement crowns and does not introduce side slopes which are difficult and dangerous to cross, particularly for vehicles turning the upper corners of the intersecting streets.

Gutters—The crown of the roadway should not be broken by a gutter across the intersecting street, which is the universal practice in some cities. Such gutters are obviously objectionable at any season, particularly so when hidden under snow and ice, and when the gutter and its approaches are so steep as to cause a violent jar to vehicles passing over them.

Any form of open gutter across a street intersection necessitates slowing up a vehicle every time the intersection is approached, which certainly is not desirable. The damage to vehicles on crossing open gutters may be considerable, especially when drivers are careless or are unfamiliar with the location of the gutters. Another serious objection is the rapidity with which the road metal disintegrates, ruts and forms depressions along the sides of the gutters. In my experience I find open gutters a source of continuous expense, if the pavement is maintained in a good passable condition.

Corner Catchbasins—Drainage at street intersections in cities which have storm-water sewers is more easily taken care of than in cities which have only a sanitary sewerage system. Storm-water catchbasins are usually constructed at the corners of the curbs, using inlets with curved faces. Such catchbasins serve well to receive surface drainage, but are objectionable because surface water collects at the corners, along the line of the sidewalk and crosswalks, and forms pools which pedestrians must jump or wade across.

Gutter Bridges—In some cities accidents and discomfort to pedestrians are guarded against by flattening the crown along the cross walk, and at several feet from the curb placing a false curb or wall with a cast-iron plate bridge over the gutter opening. While this eliminates stepping up and down at the curbs, the bridges are objectionable in that a pedestrian is liable inadvertently to step off the side and fall. Furthermore, an elevated crossing and bridge in the roadway is an undesirable obstruction to traffic; and the bridge plates are liable to frequent breakage from being struck by the wheels of heavy vehicles.

The Inglewood Method—Fig. 1 shows a standard street cross-section and Fig. 2, the plan and details of a standard street intersection of a type which has given excellent satisfaction at Inglewood, Calif. It will be seen by the elevations shown on the plan (Fig. 2) that the crown of the roadway is raised across the intersection, but not enough to be an obstruction to traffic. The line *DEF* marks the termination of the standard or regular cross-section of the roadway (Fig. 1), while the line *ABC* is the special cross-section, which begins at the ends of the gutters (see Fig. 2, *A*). *ABC* is also on the inside line of the sidewalk of the intersecting street. Between *ABC* and *DEF* is allowed a distance *G*, sufficient to make the transition from one to the other cross-section gradual and easy. I make *G* equal to the width of the roadway in feet.

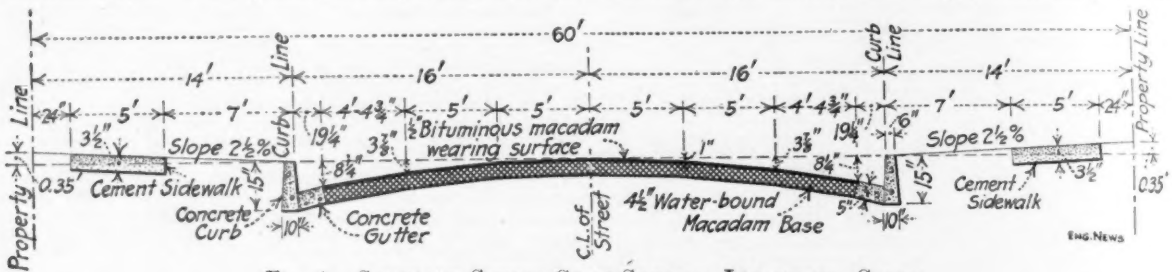


Fig. 1. Standard Street Cross-Section, Inglewood, Calif.

The crown *ABC* (Fig. 2, *A*) is determined by the following formulas in which x = the width of the roadway in feet and $x_1 = \frac{x}{8}$:

$$a = \frac{x}{200} + 0.08; \quad c = a \times 0.56; \quad d = a \times 0.25; \quad c = a \times 0.06.$$

Between the points *BB* on the center line of both streets the grade line is continuous across the intersection to the opposite *ABC* cross-sections. The transition from the *ABC* cross-section of one street to the *ABC* cross-section of the intersecting street, around the corner curb,

*City Engineer, Inglewood, Calif.

is shown in Fig. 2, *B*, which is a cross-section along the curb line, or a developed profile of the crown at the curb line. At the center point of the curb return, the crown of the pavement is flush with the top of the curb, and 1 in. higher than at the ends of the gutter. This arrangement of grades and crowns gives a good form of pavement without being an obstruction or inconvenience to pedestrians or vehicles. From Fig. 2, *B*, it is clear that pedestrians do not have to step up or down at the curbs or across a channel of water.

GUTTER AND CULVERT CONSTRUCTION—The construction of the gutters at the inlets and outlets is shown in Fig. 2, *B*. The bottom of the gutter is raised on a uniform slope (1½ in. per ft.) from a point at the inlet or outlet, as the case may be, to within 1 in. of the top of the curb, where the curb intersects the inner line of the crosswalk, as indicated in the plan by the letters *b* and *c*.

The covered culvert for carrying the drainage water across the intersecting street is shown in plan, Fig. 2, and in cross-section, Fig. 2, *C*. It consists of a concrete trough with a 36x6-in. waterway covered with a segment of a galvanized-iron culvert pipe, which rests on 1¼x1¼x⅛-in. angle irons, firmly embedded in the side walls.

The culverts under the sidewalks and parkways consist of similar concrete troughs, shown in cross-section in Fig. 2, *C*, the sizes of which are proportioned to the approximate amount of water they are to carry. The tops of the concrete troughs are made of straight sections of galvanized corrugated sheet iron of various gages, according to the span. The concrete sidewalks or parkway fills are placed directly on the corrugated tops of culverts. A cast-iron manhole frame and cover is placed at the center of the street.

The inlet construction is shown in Fig. 2, *D*. The culvert opening in the curb is spanned by a 2x2x¼-in. angle iron, which supports both the curb and the corrugated sheet-iron cover of the culvert. Whenever it is deemed necessary, I require a light cast-iron manhole frame and cover to be placed at the junction of the inlets to facilitate the cleaning out of the culvert; but this is not absolutely necessary, for the culverts can be cleaned easily from the inlet at the curb by means of a long-handled hoe.

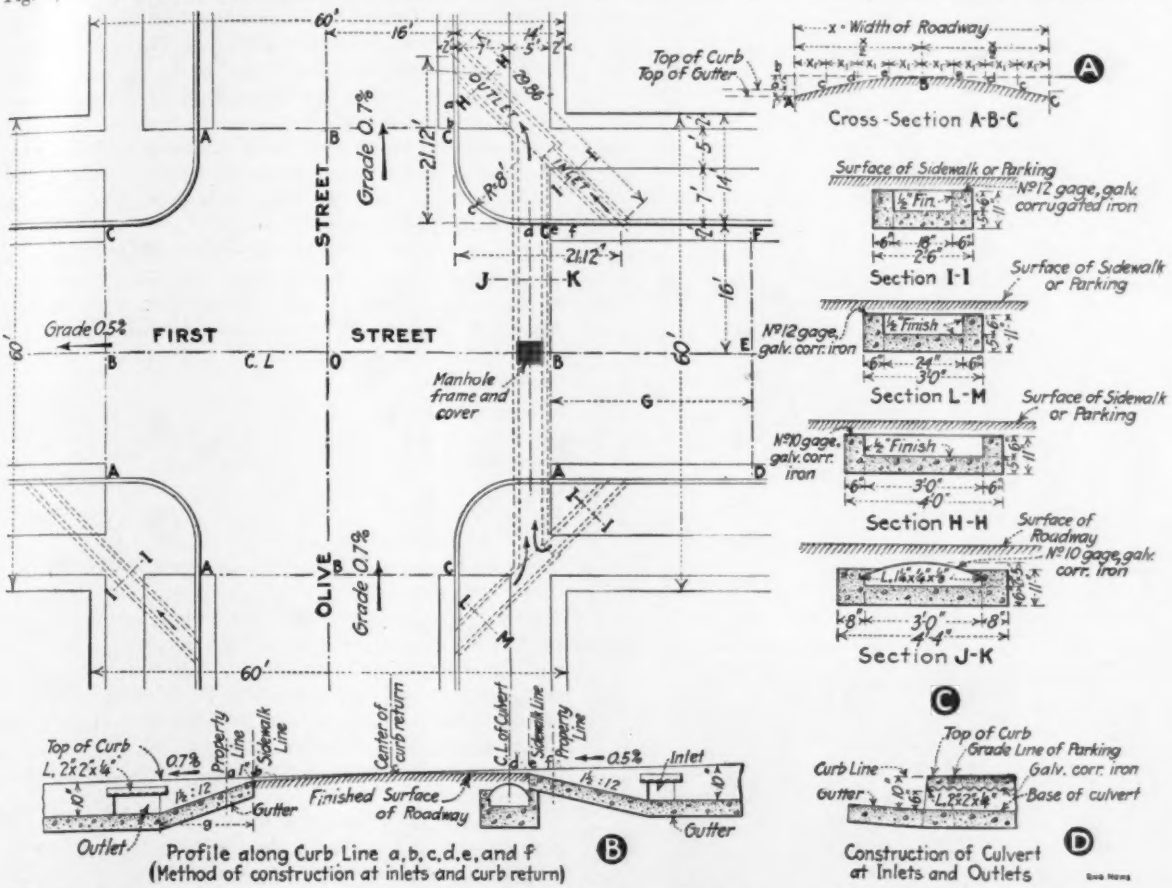
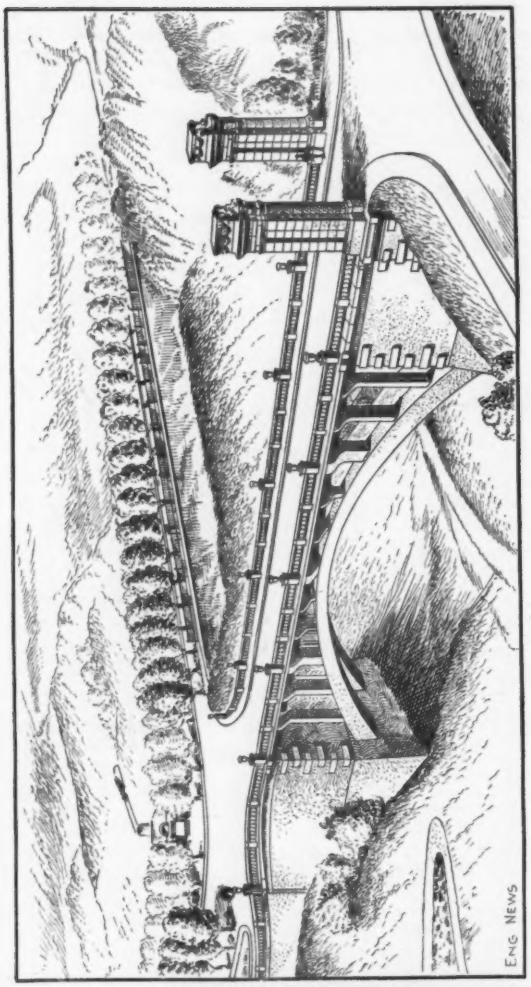
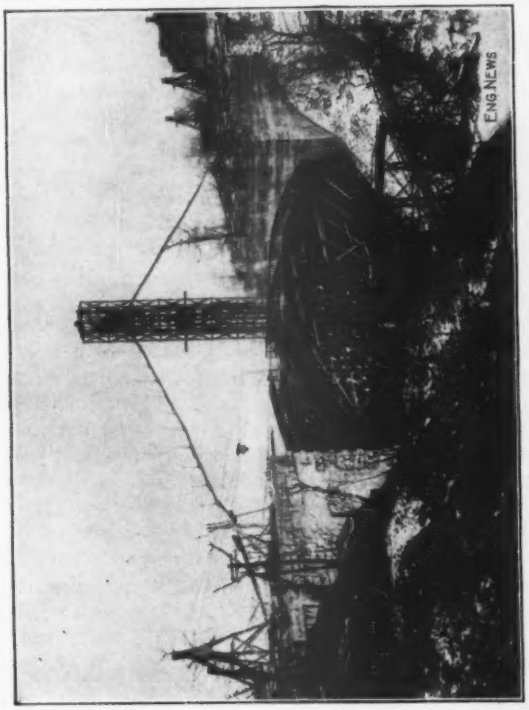


FIG. 2. PLAN AND DETAILS OF STANDARD METHOD OF SURFACE DRAINAGE AT STREET INTERSECTIONS, INGLEWOOD, CALIF.

It is of the utmost importance that the galvanized iron be of the best quality obtainable. There is no question but that the life and strength of this type of culvert depend upon the purity of the iron. Tests have uniformly indicated that segregated impurities present in iron affect its power to resist corrosion in a marked degree.

COST DATA—The cost of this type of culvert construction naturally varies with different localities and the size of the culvert necessary to properly convey the surface water. During the years 1912 and 1913, there were constructed in the city of Inglewood, 31 culverts of this design, costing complete from $92 to $275.





PRESENT CONDITION OF HAIGHTS RUN ARCH, AND PERSPECTIVE SKETCH OF COMPLETED BRIDGE
(Contractors: Cummings Structural Concrete Co., Pittsburgh.)

Progress on the Haights Run Arch, Pittsburgh

The present condition of the 216-ft. reinforced-concrete bridge over Haights Run at the west entrance of Highland Park, Pittsburgh, is shown by the photograph herewith. The character of the bridge is indicated by the sketch alongside, made from an official perspective drawing.

The abutments (4100 cu.yd. concrete) are finished, the centering and form work for the arch ribs is all in place, and the structural-steel rib reinforcement is practically complete.

The bridge is 327 ft. long, 65½ ft. wide, and 80 ft. high from floor to ravine bottom. The 216-ft. main arch consists of two parallel ribs 13 ft. thick at springing line 5½ ft. thick at crown, and 13 ft. wide; each arch is reinforced with five members made of four 6x6x ¾-in. angles spaced with radial lattice bars. The floor, of slab-and-girder construction, is divided into a central roadway 36 ft. wide between curbs, and two sidewalks 14 ft. 9 in. wide.

On account of the character of the soil and the great weight of the arches it was found necessary to carry the falsework foundations to rock, about 20 ft. below the surface of the ground. The piles used in these foundations were capped with 12x12-in. oak timbers, on which the centering posts rest.

The specifications required each of the arches to be poured in nine sections, each to be the full width of the arch and from 25 to 33 ft. long, with a 4-ft. key separating adjoining sections. The contractor therefore designed the falsework as a series of towers composed of 4 to 6 12x12-in. posts carried to within 3 ft. of the intrados of the arch. Each tower is securely braced within itself and tied to the adjoining ones. The posts are capped transversely with oak caps which serve as a base for the wedges. On account of the weight of the arches and the distance between the posts it was found necessary to use 8-in. I-beams to carry the lagging. To allow for contraction of length due to settlement of the arch, the ends of these I-beams were carried on the lower flanges of 10-in. channels set back to back, separated by 2-in. wedges.

The sand, gravel and cement brought in railroad cars is dropped into bins under the tracks and fed by gravity into a Ransome and a Hains mixer. The concrete for the abutments was distributed by chutes from two 160-ft. towers 6 ft. apart. Two light cableways, one over the center line of each arch were used to erect the falsework and structural reinforcement.

The work under contract provides for the bridge complete except the paving and railings. The quantities are about as follows: 8300 cu.yd. concrete, 800,000 lb. reinforcing material, and 3000 cu.yd. earth and rock excavation. The contract was awarded to the Cummings Structural Concrete Co., of Pittsburgh, Penn., for about $109,000, on a unit-price basis. Charles L. Steinrok is in charge of this company's work.

The bridge was designed by the Bureau of Construction of Pittsburgh. Robert Swan is Director of Public Works; N. S. Sprague is Superintendent of the Bureau of Construction, and T. J. Wilkerson is Division Engineer of Bridges. A. E. Sertore, Assistant Engineer, has direct charge of the construction for the city.

Editorials

It will doubtless come as a shock to the legal profession to learn that an Arkansas court appointed an engineer instead of a lawyer as a Master in Chancery to determine what the Arkansas Water Co. might reasonably be required to do to fulfill its franchise provisions as to quantity and quality of water supplied to Little Rock.

The findings of the case, which were briefly stated in a news note, published on p. 487 of our issue of Feb. 26, are of such a character as to make it evident that no lawyer could ever have reached them through his own unaided investigations. We have heard of a few—too few— cases where a court appointed a single engineer to investigate and submit findings; but this is the first instance we remember of an engineer serving as a Master in Chancery. We see no reason why it should not become common practice and many reasons why it should.

How large must a city or town be to afford a sewerage system? The question arises because at a recent public meeting at De Land, Fla., apparently called to consider tentative plans for a sewerage system proposed by an engineer, a state health official is reported to have declared that De Land is too small to afford sewers.

It is quite conceivable that other improvements—water-supply, for instance—should take precedence of sewers or that financial conditions may make advisable temporary postponement of a sewerage installation. It is also possible that the real or imagined financial overburden of a sewerage system is due to including in the plans sewage-treatment works which might either be simplified or omitted for the present. It should be better understood than it is, that the collection and removal of sewage from houses and towns rather than the means of final disposal, is what reduces sickness and death. This has been recognized by the North Carolina Board of Health, as stated approvingly in our issue of Dec. 11, 1913.

The progressiveness of the Metropolitan Water Board of London, England, or of W. J. Bryan, its Chief Engineer, was illustrated not long ago by the installation of Humphrey direct-acting gas pumps at the Chingford Reservoir (see ENGINEERING NEWS, Apr. 13, 1913). It now appears that the Board has also several pumps driven by Diesel oil engines, and that besides an existing 5-ft. steel water conduit, it is about to lay a 7-ft. steel gravity conduit from two new reservoirs of 3½ billion gallons (Imp.) capacity to new filters at Kempton Park.

According to the *London Times Engineering Supplement* of Jan. 21, the new reservoirs combined with existing ones at Staines and in that vicinity, and a proposed third Staines reservoir of 3.3 billion gallons capacity, will bring the London storage-reservoir system up to about 15 billion Imperial or 18 billion U. S. gal. To appreciate the enormous size of this system of reservoirs, it must be remembered that they are all artificial, built in excavation and embankment. Aside from storage reserve these reservoirs play a most important part in bacterial reduction, according to the researches of Dr. A. C. Houston, chemist to the Metropolitan Water Board.

The features noted are doubtless only a few of many which might be cited to show the wide-awake character of the administration of the London water-works.

Inserting Booster Pumps in Gravity Water-Supply Conduits

An interesting alternative to building a third water-supply conduit from Wales to Birmingham, England, has been laid before the water committee of the council of that city by Thomas Reid, of the Municipal Technical School of Birmingham. Mr. Reed proposes that the flow through the existing gravity pipe lines (two of 15,000,000 U. S. gal. daily capacity each) shall be accelerated by a series of centrifugal pumps. Booster pumps are sometimes used on force mains but we do not remember an instance of their use on gravity mains. In weighing their merits for regular service their capital and operating costs would, of course, have to be put in the balance with like figures for a duplicate or triplicate main, as also the comparative reliability of the two for uninterrupted service and quick repair in case of accident. For emergency use in case of a break in one of two or more gravity supply mains, and also for use to meet peak loads, daily or seasonal, pending the construction of a new supply main, booster pumps in a gravity conduit might sometimes save the day.

The Need for Better Highway Bridges

Obviously the conversion of all public roads into smooth-surfaced pavements is neither possible nor desirable. When the total amount of money available for road work is only a small fraction of the $5000 to $20,000 per mile necessary for a permanent road surface, how can that money best be expended?

A Canadian farmer answers the question, in the Toronto *Globe*, as follows:

> **The first need in rebuilding the roads of the country is modern bridges. The day of the wooden bridge is over. Old bridges need renewal in every direction, and there is no available timber with which to replace them. The farmer depends on thrashing outfits traveling over these roads, and accidents have already been reported in which engines have broken through. If the government has money to spare for road building, the first grants should be devoted to the building and maintaining of bridges.**

The suggestion has much sound reason back of it. The increasing use of heavy traction engines, farm machinery and automobile trucks has made the ordinary highway bridge one of the weakest links in the country road system.

It is generally agreed that if State or Federal funds are to be expended on road improvement, especially where

the funds are raised by a bond issue, they should be so spent as to achieve some permanent result. The building of permanent bridges instead of short-lived structures which require frequent repairs and which too often become dangerous because repairs are neglected, is one of the best investments of money for road improvement that can be made.

Building Government Railroads in Alaska

The bill authorizing the construction by the government of railway lines in Alaska, which has passed both houses of Congress and will probably be a law by the time this issue of ENGINEERING NEWS reaches its readers, can be paraphrased in very few words. The bill empowers the President of the United States to do practically anything he pleases in order to build railways in Alaska not more than 1000 miles in extent and at a cost not to exceed $35,000,000.

While one purpose of the railway is stated to be the connection of one or more of the Pacific harbors on the southern coast of Alaska with the navigable waters of the Yukon and with the Alaska coal fields, other clauses give the President discretion to build railways in Alaska wherever he deems their construction is justifiable.

The Senate bill provided that the President could employ any of the engineers of the Army and any engineers in civil life, but provided that any civil engineer appointed from civil life at a salary greater than $3000 per annum should be appointed with the advice and consent of the Senate. The House bill omits all such details and provides that the President may employ "such officers, agents or agencies in his discretion as may be necessary to enable him to carry out the purposes of this act." Under this broad discretion the President could, if he chose, turn over the entire work to an engineering or contracting firm. It is, of course, entirely unlikely that any such action would be taken. It is to be hoped, however, that the discretion invested in the President may be construed broadly enough so that he may place the work in charge of an able commission, made up of men of such experience and ability that the work may be carried out in an efficient and businesslike manner. Unless some such scheme of organization is adopted, there is danger that the work will be turned over to one of the departments where efficiency will be strangled with bureaucratic red tape and where political considerations will be a constant handicap to economy.

While the bill authorizes the President to operate as well as construct the railway through any agency he may select, he is also authorized by the House bill to lease the railroad after completion for terms not longer than twenty years.

A remarkable example of Congressional ideas of accounting appears in the House bill, where it is provided that any tools, equipment or other property belonging to the United States and used in the construction of the Panama Canal or other government work may be transferred to Alaska for use in constructing or operating the government railroad *and no charge shall be made therefor*. By this innocent looking clause, Congress has given authority to whoever constructs the Alaska railroad to rob every other department of the government in order to make the other departments contribute to the cost of building the Alaska railroad. On the Panama Canal work the most careful and conscientious accounting methods have been in use from the beginning, so that the final complete cost of the canal to the government might be accurately determined. The cost of all equipment purchased has been included, and to give a correct account the money received for the equipment when it is finally sold, or its appraised value if transferred to some other government work, should be deducted to obtain the complete cost. In fact, much of the Panama equipment has already been sold.

Under the clause above quoted, however, the Panama Canal appropriations, the appropriation for any job of river and harbor work in the United States, or those in connection with the Ordnance Department, the Bureau of Docks and Yards of the Navy, or of any other government bureau which happens to have tools or equipment on hand of use in the Alaska railroad work could be drawn upon for the purpose of building the railroad, to the value of the tools and equipment taken.

It is to be hoped that this clause may be struck out before the bill emerges from conference, for such confusion in government accounting and the attempt to make the Alaska work show up at a low cost which is fictitious, are in every way objectionable.

A Bill to Fetter Designing Engineers

A bill was introduced in Congress on Jan. 12 by Congressman Oldfield, Chairman of the House Committee on Patents, and in the Senate by Senator James, Chairman of the Senate Committee on Patents, which seems to us to threaten serious injury to the engineering profession if enacted.

The bill (which is No. 11,321 on the House files) is entitled "A Bill for the Registration of Designs," and its object is to provide official registration in the Patent Office for original designs applied to any manufactured product.

The bill owes its origin to the National Design Registration League, of 1309 Race St., Philadelphia, an organization formed to secure the passage of this legislation. The President of the League is R. W. Nelson, President of the American Type Founders Co., and among the members of the Advisory Committee are Charles Cheney, the silk manufacturer; John S. Holbrook, of the Gorham Mfg. Co.; Geo. H. Barbour, of the Michigan Stove Co., and Geo. K. Birge, a wall paper manufacturer of Buffalo.

The object of the bill is to enable the producer of an original design to secure what will in effect be the equivalent of a copyright upon it, so that he or his assigns will have the sole right to its use.

It is alleged that under the present law an artist or firm which produces an original design for a piece of pottery, silverware, wall paper, cloth, lace, hardware, or any other manufactured materials, has no property right in his design and it may be immediately copied by any competitor.

There is very likely reason to suppose that legislation to give some protection to such original artistic productions would be advantageous. The merits or demerits of that proposition we shall not now discuss. What we do wish to call attention to is that the framers of this

bill, in their zeal to protect original designs of this class, have gone beyond the work of the artistic designer and included the work of the designing engineer. The essential clauses of the bill read as follows:

The author of any design, original, as embodied in or applied to any manufactured product of an art or trade, or his assignee, may register such design in the United States Patent Office and obtain from the Commissioner of Patents a certificate of such registration, which shall be prima facie evidence of the exclusive right of the registrant and his legal representatives and assigns to use said design embodied in or applied to said manufactured product within all territory under the jurisdiction of the United States of America.

SEC. 2. That for the purposes of this Act, "design" shall mean pattern, shape, form, esthetic or artistic appearance, or ornamentation, or the union of any two or more of these features embodied in or applied to a manufactured product, but shall not extend to functional construction or arrangement of parts: Provided, however, that an original shape or form of a functional or operative part, which shape or form in and of itself contributes no mechanical function to said part may be the subject of valid design registration hereunder.

According to the terms of the bill quoted above, therefore, a design may be registered for any special shape or form of any manufactured product or any part thereof.

Here, for example, is a manufacturer who puts on the market a certain machine. He can register the shape which he adopts for the separate parts of the machine and will thereby obtain the right to bring a suit against anyone else who makes a machine part of the shape which he has registered. Of course, if the party sued can show that machine parts of the same shape had been used before, that will be a successful defense to the suit. But to us it seems an extremely objectionable thing that any person or firm should be given by law the right to even claim a monopoly in any feature of mechanical design where original invention is in no way involved.

The bill might well be termed, "A bill to enable the strongest concerns in an industry to drive their weaker competitors out of business." It is easy to see how such a law could be utilized to this end. Suppose a concern is manufacturing automatic scales, for example. It could proceed at once to register the shape of every part which it uses in its machines. It is of small consequence whether the shapes so registered have any originality in their design. Under the law the Patent Office is obliged to register all designs offered without investigation as to their originality. The charge for such registration is trifling. Only $1 for a three years' term, $10 for a ten year's term, and $30 for a twenty-year term. All designs will be registered originally, without doubt, for the shorter term as the law provides that the term may be extended later by payment of the difference in fees.

By the payment of a thousand dollars then, a manufacturer can register a thousand different shapes and patterns of the parts of his machines or other products. Each one of these certificates of registration is, according to the bill, "to be prima facie evidence of his exclusive right to use this registered shape, pattern or form in all territory under jurisdiction of the United States."

Truly, what a weapon to hold over a struggling competitor!

In the conviction a year ago of officers of the National Cash Register Co. on the charge of driving out of business their rivals by the use of unfair methods of competition, it was shown that the Cash Register Co. maintained a "gloom room" through which they escorted the officers of competing companies to show them how other concerns who had tried to break into the cash register business had met disaster.

If Congress enacts this apparently innocent bill for the registration of original designs, we may expect to see the Federal courts converted into most effective "gloom rooms." Any concern which seeks by unfair means to injure its competitors can easily bring suits by the dozen or the hundred against such competitors for infringement of its designs; and will be thus aided with the processes of law to prove to its rivals the dangers of "bucking the trust."

Nor is the mechanical engineer and machine builder alone affected by this proposed law. "Manufactured product," the term used in the bill, is broad enough to cover many things with which the civil engineer has to do. There is trouble already among engineers engaged in bridge design because certain patents have been issued on concrete construction which are claimed to be void for lack of novelty, but which yet stand as a barrier in the way of the independent bridge designer who has no money with which to defend a patent infringement suit, no matter how weak the patent on which the suit is brought.

But with this Design Registration bill a law, there would be thousands of designs registered for every part and piece of a concrete bridge and a steel bridge; for concrete sidewalks and retaining-walls, for dams and conduits and docks, for every line of manufactured product made under civil engineers' direction and for every tool used by civil engineers.

It may be said that reputable concerns with upright management would not adopt such methods; that they would only take out design registration on work of real originality deserving such protection. But if such concerns would not misuse such a law they have plenty of competitors who would do so. What an opportunity for the promoter of blackmailing litigation and for the fraternity of ambulance-chasing lawyers to be able to obtain at the small price of a dollar, a document giving them the right to bring a suit in the Federal Court for infringement with a claim for large damages!

But the argument may be made that there is a real need for such a law to protect original work by the designing engineer. A concern spends a considerable sum for example on the design of a new machine or material, in which there is nothing of such absolute novelty that a valid patent could be obtained, and which has yet involved a large cost to produce. Any rival firm can make an exact reproduction of this machine or product as soon as its market value is demonstrated. It is urged that the firm spending money for such original design should have some protection against such copying.

It may be freely admitted that such copying does occur, and that there is more or less injustice in it; but let us not forget that the attempt of which we see so much nowadays, to remedy each and every evil under the sun by the passage of legislation is bound to result in failure from the very nature of things.

There are many cases where it is truly far "better to bear the ills we have, than fly to others that we know not of."

It is far better for civil and mechanical engineers and for the manufacturers doing business in this field to leave the art of mechanical design free and open. While, as we have pointed out above, the great concerns might

make use of the proposed law to crush their competitors, they might also find that it would prove a two-edged sword. Their own designing staff would find themselves hampered at every turn, and the defense of suits brought by more or less unprincipled blackmailers might involve heavy expense. Whatever else may be done with the pending bill, therefore, we hope that it may at least be so amended as to confine it solely to artistic designs. There is no apparent reason why any other designs involve such originality or expense in production that the grant of a monopoly of their use for three or ten or twenty years can be in any way defended.

It is especially noteworthy that Chairman Oldfield, who introduced the bill in the House, is the sponsor for the radical measures introduced in the last Congress to emasculate the system of granting patents on inventions. But the abuses against which that legislation was aimed would be increased a hundredfold under such legislation as that contained in the pending bill. Certainly a Congress which is attacking trusts and monopolies so vigorously ought not to enact such a measure.

Letters to the Editor

Relative Capacities of Intercepting Sewers, Pumping and Treatment Plants

Sir—W. H. Dittoe, chief engineer of the State Board of Health of Ohio, inquires in your issue of Feb. 12 whether the capacity of interceptors should not be limited to the ultimate maximum dry-weather flow of sewage to be pumped and treated, and the capacity of the pumping and treatment plants be made sufficient for the maximum flow of the interceptors "thus determined" and enlarged as the tributary population increases until the ultimate population is reached.

In the opinion of the writer the entire plant should be designed with the maximum ultimate demands that may be made on it kept in view, but this does not mean that it should all be constructed in the beginning.

As to the interceptor, it is customary to build this of the ultimate capacity, or at least sufficient for a long term of years, because differences in diameter affect the cost to a much less extent than does duplication of the work; but where the ultimate sewage flow is in much doubt or where the accumulated fixed charges for the larger interceptor would exceed the cost of duplication by the time this is required (due regard being given to the difference in grade as affecting costs of excavation and pumping) then it is better to construct the interceptor of a capacity sufficient for a moderate term of years in advance, leaving a second line to be built when required.

Such a course is of especial advantage where a great increase in flow is expected in the future, as provision for adequate velocities during the early years of use can be more readily provided by the smaller-sized sewer, although this difficulty may often be met by a special design of the invert to adapt it to the minimum flow. Yet another case is where, on account of the great size ultimately required, it seems best to construct two conduits. In this case, it would, of course, be unwise to build more than one at the outset. As a general proposition, however, it is considered good practice to put in the interceptor once for all of the maximum capacity required by flows forecasted from 30 to 50 years ahead.

Mr. Dittoe inquires whether it is "not illogical to provide a capacity for the intercepting sewer *in excess* of the volume ultimately to be pumped and treated." In a general way this is true; but in the first place a certain margin—say 25 or 30%—is usually allowed in sanitary sewers to provide against extreme unforeseen fluctuations that might produce an undesirable internal pressure on the sewer barrel and cause a backing up of the sewage into basements; and a further allowance may sometimes be sanctioned in view of the unknown character of future development in the tributary territory. A few large hotels or office buildings might render a sewer designed on too narrow a margin entirely inadequate. For these reasons the size should not be too restricted.

When it comes to pumping and treatment plants the case is different, as these may be installed by units at a cost not greatly in excess of complete initial construction—usually at a very material saving in fixed charges. But in these cases it is usually best to secure land sufficient to fully cover future requirements, as this expense must be met sooner or later and in all probability will amount to less now than hereafter.

KENNETH ALLEN,
Engineer, Metropolitan Sewerage Commission of New York.
17 Battery Place, New York City, Feb. 16, 1914.

The Preliminary Investigations of the Stony River Dam Foundation

Sir—In your issue of Feb. 12, 1914, you published a letter in which I explained my connection with the Stony River Dam. Since then there have been published two statements which I would like to correct.

In a Bulletin on "The Facts as to the Blowout under the Stony River Dam, at Dobbin, W. Va.," just issued by the Ambursen Co., of Boston, the statement is made that Mr. Ambursen, the engineers of the owner, and myself as Consulting Engineer, visited the site of the dam, early in April, 1912, and examined jointly the test pits.

I am informed that the Quebec Streams Commission has a report on the partial failure of the Stony River Dam, in which the engineers mentioned above and myself are said to have met at the dam during the construction and to have determined the depth to which the cutoff wall was to be built.

Both of these statements are absolutely incorrect. I

visited the site of the dam only once and that was in the Fall of 1911, about six months before the construction was begun. R. P. Bloss, the engineer of the owners, was the only person who accompanied me.

Based upon the indications of the test pits and borings, Mr. Bloss and I prepared a plan to be submitted to bidders, showing the probable depth of the cutoff wall, which was to be 7 ft. at the ends and 10 ft. when the depth of the water became 25 ft., and was to be carried down to rock in the center of the valley. The actual depth of this wall had, of course, to be determined during the construction. At each end of the dam we showed sheet piling.

As actually built, the cutoff wall was made only 5 ft. below the floor of the dam for about 200 ft. and then carried down to rock. No sheet piling was used, on account of difficulty in driving.

The final plans for the dam, prepared by the Ambursen company were never submitted to me and my services ceased before the construction was begun.

EDWARD WEGMANN.

New York, Feb. 22, 1914.

The Hydraulics of Canal Lockage

Sir—Some time ago I called your attention to an erroneous statement in your columns (which I later learned had been copied from *The Canal Record*, in regard to the effect of the presence of a boat in a lock chamber upon the quantity of water consumed in filling or emptying the chamber.*

Quite a number of inquiries have since come to me for an explanation as to how it can be that the boat makes no difference in the quantity of water consumed. The contrary view is so plausible at first thought that it is easy to become confused. Moreover, there are certain features of the passage of boats through locks in which the size does make a difference in the consumption of water, and failure to differentiate cases properly doubtless adds to the perplexities of the question. It may be of service to those of your readers who find difficulty in getting a clear conception of the problem to present the following analysis:

Referring to the accompanying sketch,

Let Q be quantity of water in lock chamber when filled to higher level.

Let q be quantity of water in lock chamber when filled to lower level.

Let P, P', P'' be the prisms of lift, or the difference in quantity of water in lock chamber between higher and lower levels.

Let D, d, etc., be the displacement of boats of different sizes.

Then in (1)

$$P = Q - q$$

In (2)

$$P' = (Q - D) - (q - D) = Q - q = P$$

In (3)

$$P'' = (Q - d) - (q - d) = Q - q = P$$ and so on.

That is, the value of D, d, etc., makes no difference in the value of P, a conclusion which is graphically obvious from a comparison of the shaded portions in Figs. 1, 2 and 3, which are manifestly equal to one another. Therefore, the presence or absence of a boat, or the magnitude of its displacement, makes no difference in the quantity of water required to be introduced into, or withdrawn from, a lock chamber to change the surface from one level to another.

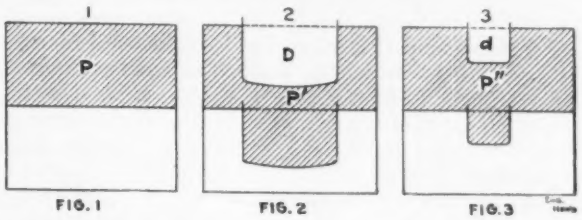


FIG. 1 FIG. 2 FIG. 3

But if it is a question of the quantity of water which is transferred from the upper to the lower pool in passing a boat through a lock, the size of the boat does make a difference. A little analysis will show that a descending boat causes a transfer of water from the upper to the lower pool equal to P-D, P-d, etc. The larger the boat, therefore, the smaller the transfer of water.

If D becomes as large as P, there would be no transfer at all, and if larger, the transfer would actually be negative; that is, it would be from the lower to the higher level; and under this hypothesis the movement of boats downstream exclusively would result in a flow of water upstream. This anomaly, however, is of the very rarest occurrence, for almost always P is greater than D. But occasions do arise in locks designed for the passage of rapids on rivers where in flood time the difference of level between the upper and lower pools so nearly disappears that P may actually be smaller than D.

In the case of upstream passage, the transfer of water, as will readily be seen, is $P + D$, $P + d$, etc. From this, it appears that the quantity of water consumed in passing a boat up increases with the size of the boat. If the number of passages up equals the number of passages down for any size of boat (the normal condition, if a considerable length of time is considered), D, d, etc., on upbound traffic balance off these quantities on downbound traffic, and thus again the size of the boat makes no practical difference.

H. M. CHITTENDEN.

843 Central Bldg., Seattle, Wash., Jan. 28, 1914.

A Question on an Examination Question

Sir—An examination for the position of Superintendent of Construction, Department of Highways, was held last summer by the New York State Civil Service Commission. The following is one of the examination questions:

Assume that an old highway bridge truss, span 35 ft., set on masonry abutments with floor 10 ft. above stream bed, had been washed out in a freshet.

Making any further assumptions needed, prepare an estimate for replacing this bridge by a 6x6-ft. culvert. Include every considerable item that would affect the cost of materials and work.

I concluded that this problem had been given to determine the practical common sense of the applicants and so made a rough, liberal estimate, including road repairs and all other considerable items of expense which would

*See "Engineering News," Oct. 16, 1913, p. 781, and Nov. 6, 1913, p. 937.

be involved. To this estimate I appended a statement substantially as follows:

To the foregoing amount there should be added a sum of money which, when capitalized at 4%, would produce an annual interest equal to the above amount, since the culvert would have to be replaced annually. In other words, this is a ridiculous problem. Instead of replacing a 35-ft. truss span with a 6x6-ft. culvert, a longer truss or girder span should be provided.

After the examination papers had been graded, I found that I received a bare passing mark on this question. An inquiry brought forth the reply that the estimated cost was too high. The representative of the Civil Service Commission, when interviewed, defended the problem as being entirely practical, stating that it was easily conceivable that the masonry abutments of a bridge might have been undermined in a freshet which could be taken care of by the proposed 6x6-ft. culvert.

I, however, am so obtuse that I cannot conceive how a stream which will destroy a bridge under which it passes will flow safely through an opening having approximately one-tenth of the cross-sectional area provided for the original structure.

"SUBSCRIBER."

Albany, N. Y., Feb. 19, 1914.

[It is true that county highway bridges sometimes have their abutments undermined by a freshet not high enough to reach the bridge itself. But to replace a 35-ft. bridge span by a 6x6-ft. culvert means that the culvert would flow full when the stream under the bridge was only a foot deep. It is not at all strange that those taking the examination thought there was some catch in the question.—ED.]

The Autoclave Test Not New

Sir—In your issue of Feb. 19, 1914, p. 426, there is a reprint of H. J. Force's patent on the autoclave test for cement.

This method of testing cement was thoroughly tried out in Germany in the years 1875 to 1896, and the German cement literature of these years has many discussions pertaining to the advisability of incorporating this test in the specifications of the Verein Deutscher Portland Cementfabrikanten. A patent was issued to Dr. Michaelis, Oct. 5, 1880, which he termed his high-pressure steam test. By this test briquettes, neat and 1 to 3 sand were subjected to 24 hours curing in a damp closet, then to steam pressures varying from 10 to 30 atmospheres. Dr. Erdmenger brought forward about the same time his high pressure boiling test which was different from the former only in placing the briquettes in water and then subjecting them to the pressures given above.

In 1898, this test was in use in the laboratories of the Wolverine Portland Cement Co., and is probably still in use there, all cements before shipment being required to pass this high-pressure test.

Dr. Oscar Gerlach, who had used this test in Germany, instituted this method of testing in this country and is directly responsible for many cement laboratories in the United States being equipped with apparatus for high-pressure steam test of briquettes, the testing being carried out for soundness and accelerated tensile strengths. The pressures used in these high-pressure containers varied from 90 to 100 lb. The pressures most in use in Germany in the period 1875 to 1896 varied from three to twenty atmospheres. See "Thonindustrie Zeitung," 1893, p. 898, also "Thonindustrie Zeitung," 1892, p. [illegible]. This article states:

It is proposed to carry out the test for soundness not in cold water and not in hot water, but to carry it out by the use of high pressure, using from three to twenty atmospheres. Dr. Erdmenger asserts that with this test every cracking of the cement can be uncovered which is directly due to the presence of free lime or lime not combined with silica or magnesia.

After a number of years of discussion this test was rejected by the Verein Deutscher Portland Cementfabrikanten as being inconclusive and liable to cause the rejection of sound cement. Dr. Michaëlis advocated the rejection of such a test largely on the ground of the chemical reactions occuring at these pressures and atmospheres being unlike anything occurring in nature in masses of portland cement concrete.

H. E. BROWN.

52 East 41st St., New York City.
Feb. 24, 1914.

[In view of the above information, more or less of which has long been known to cement experts, it has been suggested that Mr. Force could not get a patent on the autoclave test. This reminds us of an incident in connection with the recent impeachment of Gov. Sulzer, of New York. In the investigation prior to bringing the impeachment charges, the New York Assembly summoned as a witness one Garrison, a former New York City newspaper reporter who had been unofficially acting as press agent for the Governor. Mr. Garrison had a very limited respect for the august Assembly and promptly and conclusively refused to answer a number of leading questions put by the Counsel in charge of the investigation. This refusal so offended the Assembly that it took advantage of its then judicial nature and handed him over to the warden of the Albany prison under charge of contempt of court. After he had been there for about three days, in precisely the same condition, so far as his personal comfort was concerned, as though he had been caught in the act of burglary, he received a telegram from his fellow reporters in New York. It read "They can't put you in jail for that."

It is, however, one thing to obtain a patent for an alleged new invention and another and very different thing to sustain the validity of the patent in court in an infringement suit. Interested readers should carefully study the two brief claims in Mr. Force's patent. The first is for a method consisting in boiling cement test pieces at a pressure in excess of 50 lb. per sq.in. The second is for a method consisting in boiling them under a pressure approximating 285 lb. per sq.in. If these methods are not novel, the patent is invalid.—ED.]

Combined Bending and Compression in Derrick Booms

Sir—In your issue of Feb. 19, E. Godfrey presents a method of computing the maximum stress in a derrick boom subject to direct compression and bending due to its own weight, which takes into account the effect of deflection. A simpler method of arriving at this stress is as follows:

Let

W = the weight of the boom;

P = the compressive load, applied centrally at the ends;

L = the length of the boom;

M = the total bending moment at the middle;
Δ = the total deflection at the middle, supposing the boom to have been straight in the beginning.

Then

$$M = \frac{WL}{8} + P\Delta$$

Assuming, as Mr. Godfrey does, that the boom in deflecting takes the form of a parabola, it follows that the bending moment due to the compressive load has a parabolic distribution along the boom, the same as the moment due to its own weight. We may therefore write

$$\Delta = \frac{5\,ML^2}{48\,EI}$$

and, substituting this value above

$$M = \frac{WL}{8} + \frac{5\,PML^2}{48\,EI}$$

From this we derive by simple algebraic transformation

$$M = \frac{WL}{8 - \frac{5\,PL^2}{6\,EI}}$$

With M determined, the rest of the process involves only the familiar

$$f = \frac{P}{A} + \frac{M_c}{I}$$

Working through Mr. Godfrey's numerical example with the aid of the above formula, I check with him exactly provided he used for E a value of 1,500,000 lb. per sq.in.

It is worth while to note the connection between the formula derived above and Euler's formula. Suppose P increases until the denominator in the above formula becomes zero; i.e., until

$$8 - \frac{5\,PL^2}{6\,EI} = 0$$

Then

$$P = \frac{48\,EI}{5\,L^2} = \frac{9.6\,EI}{L^2}$$

Euler's formula is

$$P = \frac{\pi^2 EI}{L^2} = \frac{9.87\,EI}{L^2}$$

The close agreement noted is sufficient justification of the assumption for all practical purposes of a parabolic deflection curve adopted by Mr. Godfrey.

JOHN R. NICHOLS.

Cambridge, Mass., Feb. 23, 1914.

Self-Acting Dry Reservoirs for Flood Prevention

Sir—As a method of controlling floods in our northern streams the self-acting dry reservoir system seems to be the most adequate for efficient control and durability and is least objectionable during the low-water period.

Such reservoirs should be built as near the source of the rivers as practicable, thus giving protection to all until this system gives way to the present levee method, which has given good satisfaction on the lower Mississippi for many years, and which is the only method that can control floods in that part of the valley.

By self-acting dry reservoirs, I mean a reservoir that is never entirely closed and will empty itself at a certain calculated rate of flow without the aid of man or machinery, and will, in a calculated time, empty itself and leave the area occupied by the water from flood storms dry and ready for a return of the same conditions.

The necessity for self-acting reservoirs arises out of the uncertainty of man to perform public duties during times of great storms. Machinery should not be depended on because of the possibility of it being out of repair when needed and the time necesarily required to make changes, and the uncertainty as to the quantity of water each operator would release, when possibly in an excited mental condition.

The dams for self-acting reservoirs should be built so high that the water from any of the great floods on record would not fill them to more than 75% of their capacity, thus leaving a safe margin for a greater runoff than any yet recorded.

As regards safety and cost of repair, the dams forming self-acting reservoirs would be superior to levees for flood protection, with their great length and the many dangers to which they are subjected.

H. E. F.

Are Country Roads so Bad as Represented?

Sir—As Congress is now debating the spending of $25,000,000 a year by the national government in aid of road building throughout the country, this seems a proper time to present some data not heretofore published except in a scattered way as to the actual degree of badness of unimproved roads—exaggeration eliminated so far as possible.

In the book on Public Roads, issued by the Missouri Bureau of Geology and Mines, published Feb. 15, 1907, Chapter VII, p. 102, is a table showing the condition of Missouri post roads in 1906. In answer to the question, "How many days in the year are the roads 'impassable'?" reports from 100 rural mail carriers averaged by the writer give 29 days a year. If these reports are biassed, the bias is likely to be adverse—strongly adverse—to the unimproved roads; and 97.5% of the roads of the state are unimproved according to same book. For in interpreting the word "impassable" the carrier is likely to construe it not literally but as meaning bad or very bad.

Now if those whose bias is likely to be against the existing roads report them as being, we will say, quite bad only 29 days in the year, is not the offhand statement that the roads are bad six months in the year an exaggeration, and the assertion that they are utterly bad the whole year round still further from the truth?

In yesterday's daily paper (Feb. 6) is an Associated Press dispatch from Antwerp, Belgium, giving statistics of the increase of the national debts of the world, which states that those debts have increased sixfold in the last century.

Is not this of significance when proposals to spend $25,000,000 a year on roads by the national government (besides what the states and counties spend) are under discussion?

The writer has just returned from a 20-mile trip over mud roads in this state (Tennessee). The roads were bad, I grant, but not so extremely bad but that we could

get along and trot the horses a good part of the time; and this is mid-winter when roads are worst.

Let the truth without exaggeration either way be told about our bad roads—and also about our "good roads," which after being built only 3 or 4 years are giving out and going to pieces.

A. P. C.

Nashville, Tenn., Feb. 7, 1914.

Computing Devices for Astronomical Problems

Sir—It is obvious from the description of the "Mechanical Navigator," in ENGINEERING NEWS, of Jan. 22, that it is a combination of the well known solar attachment and the ordinary engineers' transit, recalling the ordinary attachment riding on the transit telescope, when the telescope is pointing due south, while the solar is pointing at the sun, and comparing this to the illustrations of the "Navigator," the similarity becomes evident.

Thus, "plane *A*" corresponds to the equatorial plane in which the transit telescope lies, the angles in this plane being measured by an hour disk, usually found in the old-style solars; the "hour circles *B* and *B'*" correspond to arcs on the solar drawn from the tip of the solar, which points due north, to the hour disk; these arcs are not usually built, being unnecessary. The arcs "*C* and *C'*" correspond to the ordinary vertical arc of the transit; "*D*" corresponds to the ordinary horizontal arc of the transit.

The only basic point of difference is that in the "Navigator" the fixed elements are the equatorial and the north pole, while in the surveyor's transit with solar attachment the fixed elements are the horizontal plane and the zenith. Because of this difference the transit is capable of both measuring and computing, while the "Navigator" can only compute the spherical triangle.

For the mere mechanical computation of the triangle this ingenious machine, about 4 cu.ft. in volume, and $2400 in cost, appears to the writer a *reductio ad absurdum*. The problem solved, as an example, in your description of the "Navigator" is solved as accurately or more so, on the "Meridiograph,"* which is only a 7-in. disk, and not much thicker than a piece of cardboard, by a *single* setting for *apparent time*, by a *double* setting for *azimuth*—as rapidly as the reading of a slide-rule.

Instruments such as the solar or the "Navigator" consist, in the simplest analysis, of seven axes, standing on end and rotating with reference to each other, the angles between these axes being laid off equal to the altitude, latitude, hour angle, azimuth, declination, and 90°. The arcs on which these axes rotate are only a few inches long. High mechanical skill is used to adjust the axes so perfectly that the last one of these axes will give an accuracy of 1′ or 2′.

The engineer accustomed to reading verniers to 1′, who appreciates that on a 6-in. disk 1′ means a space $^1/_{1200}$ in., or one-half the thickness of ordinary tissue paper, will appreciate the task of adjusting seven axes, all rotating with reference to each other, so perfectly that the resulting accuracy will be 1′ or 2′. A stiff breeze, or a touch of a twig would seem enough to upset the fine factory adjustments.

*Described in "Engineering News," Feb. 26.

The ordinary scale graduated to $\frac{1}{10}$ in. can be read with a fair degree of accuracy to $^1/_{100}$ in.; the ordinary 10-in. slide-rule, when read to the limit of interpolation—4 significant figures at left end, and 3 figures at the right end—requires a measurement by eye of $^1/_{250}$ in. Taking the average of these as the ordinary capability of human eye, it requires a 3- to 7-ft. circle to lay off angles to 1′, legible without micrometers. By using mathematical scales, such as I have used on the "Meridiograph," I attain on a 7-in. disk a magnitude and openness of scales equivalent to that of circles 3 to 12 ft. in diameter.

Louis Ross.

268 Market St., San Francisco, Calif.,
Jan. 30, 1914.

A Filipino Civil Engineer

Sir—Those of your readers who believe there should be a legal definition of the term "engineer" may find some ammunition in the article by Dean C. Worcester, entitled, "Our Fifteen Years in the Phillippines," in *Munsey's Magazine* for February, in which the accompanying cut and caption appear.

Reading and writing are evidently not necessary to an expert road and bridge engineer, in the Philippines.

The caption reads as follows: "This Bukiduan, Though Unable to Read or Write, Has Become an Expert Road and Bridge Engineer." The qualifications of an engineer seem to be less exacting in the Philippines.

P. M. LA B.

Chicago, Ill.
Jan. 22, 1914.

Employment and Investment Combined

Sir—Referring to the article in ENGINEERING NEWS, of Feb. 19, signed X. Y. Z., and the editor's remarks thereon, I would say that the article evidently refers to the United Engineering & Construction Co., of Cleveland, Ohio. This matter is now under investigation by the post office authorities and has been since last December, also by the authorities of Michigan and other states.

The instigator of the swindle was Theodore Rodman, who claims to be the son of the richest zinc-mine owner in Norway and claimed that he would finance the undertaking. He also claims to have been an engineer with the Westinghouse Co., Pittsburgh, the Henry P. Worthington Co., the Otis Elevator Co. and others.

W. R. Rose, the President, claimed to have been General Manager of the Monahan Construction Co., etc. Is it any wonder that a number of experienced engineers and

architects thought the undertaking to be based on substantial and honest principles and believed it to present a good business opportunity?

The scheme of the promoters was to launch eventually a $500,000 or $1,000,000 company, to sell out the lion's share of the stock which they practically annexed for their services and decamp, but thanks to the authorities in Columbus, the greater swindle was nipped in the bud. A similar swindle perpetrated in Pittsburgh by the same parties about a year ago has also been unearthed and is being re-investigated. I have taken the liberty to inform the Secretary of the American Society of Mechanical Engineers and I hope that a warning against similar schemes may be issued by the society.

I am giving you these details for the benefit of the engineering profession. Engineers are, as a rule, honest men. Their business instills this trait in all their dealings and no doubt for this reason the promoters succeeded in duping about two dozen engineers and architects.

EDW. F. NEUKOM,
Mechanical Engineer.

35 Central Ave., Dayton, Ohio, Feb. 28, 1914.

[The above refers to the letter published on p. 425 of our issue of Feb. 19. The warning against investments of this sort printed as an editorial in our issue of Nov. 13, evidently had the effect we intended in cautioning engineers, for we at once received very angry letters and telegrams from Rose and his associates.—ED.]

The Ohio Flood Prevention and Conservancy Act

Sir—The Ohio Flood Prevention and Water Conservancy act contemplates, as per your issue of Feb. 5, the establishment of conservation districts "within, or partly within and partly without, any county in the state." Your comment, that conservancy districts might better be natural drainage areas, is well supported by facts, most particularly so in the case of regulation of flow by storage.

The drainage areas of most rivers, large or small, are of wide expanse at headwaters and narrower down stream. A river on which regulation by storage is economically practicable is one along which there are enough natural storage basins, located sufficiently far below the sources of the stream, to give enough runoff to fill dischargeable reservoirs to effect the required minimum of flow. The total works for such a project may be far away, several counties away, from the mainly benefited area, densely populated cities and towns down stream, and be of little benefit at their actual location, where condemnation proceedings for reservoir area would, quite probably, be strongly opposed. The downstream municipalities, the sufferers from the river in flood, would be eager to form a conservation district, including their own territory, of course, and, as to that, there appears to be nothing in the Act that would prevent.

I have in mind a stream on which there is an apparently good site for a large storage reservoir, fulfilling the varied requirements of such a site, at the downstream edge of a headwater county, while the most benefited considerable towns and cities are far downstream.

In the great project of the Pittsburgh Flood Commission, there is practically the same condition on a large scale. The county in which is the city of Pittsburgh contains not one of the projected 43 reservoirs, which are well up the two converging rivers and their tributaries; mostly many miles distant.

The situation might well be that the counties of the reservoir sites would refuse to join the conservation district, thus running risk of government, with large powers of assessments, by municipalities far away.

A river does not pertain to any municipality, or group of municipalities, of its drainage area, alone. Change in hydraulic conditions will carry down stream. A wide overflow of river banks gives a reservoir condition which will aid more or less in retarding the downstream flow. Material increase in velocity at any point, such as is caused by an improved, confined channel, will give more rapid delivery and therefore more rapid piling up of flood waters downstream, and for such effort the causer must be held responsible.

The conservation district to comprise all the drainage area required for a whole river project, and a central state authority, appeal as the rational method.

W. H. BREITHAUPT.

Berlin, Ont., Feb. 26, 1914.

The Elimination of All but Two Types of Road for country highways is recommended by Robert Earl, who was a member of the New York State Highway Commission during Governor Hughes' administration, 1909 to 1911. Mr. Earl was recently a witness before an investigating commission of the State Assembly, and his testimony is reported in press dispatches as follows:

Mr. Earl believes that only two types of roads should be constructed in the future, judged by past experience—brick roads to cost about $25,000 a mile, and dirt roads made at a cost of $2000 and $3500 a mile, to be kept in good repair each year. He is not friendly to the macadam roads which cost from $12,000 a mile to $16,000 a mile, insisting that they do not stand up any better than his proposed improved dirt roads. He doesn't believe in building any type of road at a cost between his dirt road and a brick road.

The Relation of Municipal Utilities to the municipality is discussed by the Wisconsin Railroad Commission in the recent Sparta water-rate case. The city had applied for authority to increase and readjust its water rates. The examination of the Commission showed that the water-department accounts had not been kept so all transactions could be clearly disclosed, although the water commissioners and city officials were agreed that the department ought to be put on a strictly business basis. The Commission laid down three fundamental rules which should be followed by municipal plants in general:

(1) "The city must pay the utility at a reasonable rate for all services rendered." Thus water for fire purposes and street sprinkling, and electricity for illumination of streets and buildings, is to be paid for out of general funds. Private consumers should not bear any part of this burden, except as taxpayers.

(2) "The utility must pay the city a reasonable amount for interest on city equity and for taxes." The city's investment in water-works plant, if properly invested in some other way, would bring a return in form of interest and would consequently decrease the city rate. Unless the city is paid a reasonable amount for interest, the rates for private users will be less than the cost of serving them. A reasonable amount should also be paid for taxes if discriminations, against taxpayers in favor of private consumers, are to be entirely eliminated.

(3) "The city must provide all funds for capital expenditure, such as new extensions, retirement of bonds, etc." A clear distinction must be made between expenses of furnishing service and other expenditures. It often happens that extensions are made by using cash representing offsetting assets for depreciation reserve; this postpones the time when the amount would have to be provided by the city through taxation or sale of bonds. This investment is merely temporary and the cash required for replacements must be provided by the city and the city equity increased proportionately.

Whether or not the element in city equity, resulting from appreciation of plant value and reinvestment of profits, should be included in the amount upon which interest is to be earned is largely a matter of local policy.

Consulting Engineers' Report on Concrete Work, League Island Park, Philadelphia

League Island Park, a civic improvement in the city of Philadelphia, Penn., has been under construction since 1899. Already approximately $1,500,000 has been paid out exclusive of moneys for the acquisition of land, and an additional $1,000,000 is yet to be expended. Of this amount, about $275,000 has been paid out for concrete work, including a wall around a lake or lagoon, a considerable length of concrete sidewalk and three concrete bridges. All of this concrete work is now in varying stages of disintegration, and is obviously not in as good condition as would have resulted from correct design and construction. In recognition of this fact, the Director of Public Works of the city, Morris L. Cooke, appointed last October a board of experts, consisting of Frederick W. Taylor, Richard L. Humphrey and Allen Hazen, to look into the circumstances surrounding the design and construction of the concrete work at the Park, and to report upon these circumstances in relation to the present condition of the work, all of which was done under a contract dated May 21, 1909, between the city of Philadelphia and Edwin H. Vare. An abstract of this report is given below:

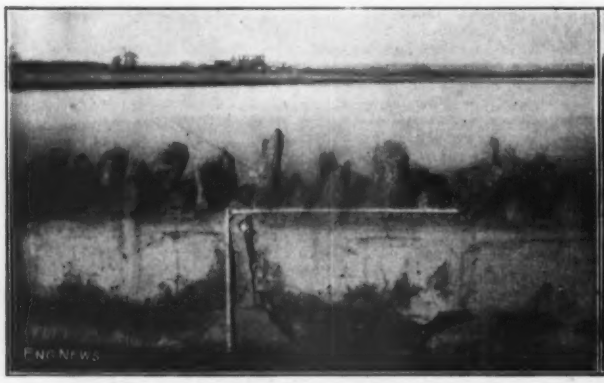


FIG. 1. INSIDE OF CONCRETE WALL, LEAGUE ISLAND PARK, SHOWING CRACKS IN CONCRETE. LAGOON IN BACKGROUND

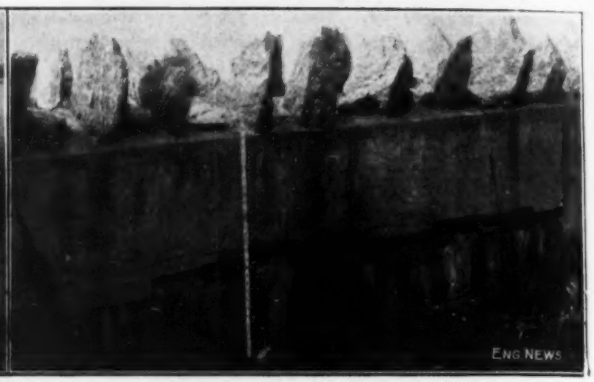


FIG. 2. NEAR VIEW OF FISSURED WALL

Sir:

In conformity with instructions the undersigned committee investigated the work at League Island Park and the following is a summary of its findings:

The sum of $277,727.10 has been paid for concrete work. The work was badly designed, badly built, and the result is almost useless to the City of Philadelphia. The course pursued in authorizing and carrying it out is that which would be expected should the main object be to create profitable work for a favored contractor.

We are forced to this conclusion by the following facts which have come to our attention:

1. The concrete work was not only badly designed but,—
   Should never have been built,
   Can serve no useful purpose, and
   Will probably have to be removed before a satisfactory Park can be made.

2. The plans and specifications upon which bids were asked were so incomplete, indefinite and vague that from them no bidder could possibly tell what would be required.

3. The scandalous method of advertising was plainly for the purpose of preventing competitive bids.

4. The time between advertisement and opening of the bids was so short as to render it next to impossible to obtain intelligent bids.

5. As the result of this procedure only one man bid on the work.

6. The contract was awarded to a sole bidder at a high price.

7. This contract was extended by supplementary agreements practically doubling the original amount without asking for bids.

8. The loose provisions of the specifications open the way for a contractor to do bad work.

9. The whole scheme of inspection was of the worst type. The contractor was required to furnish and pay an inspector who nominally protected the city's interests. Under this system it is not to be supposed that any real protection would be afforded.

10. Parts of the work were badly built and did not comply with the specifications.

11. In our opinion the bad portion of the wall will continue to grow worse. In any case the greater part of the wall should be removed for the proper development of the Park. The sidewalks cannot be satisfactorily repaired and will have to be replaced.

The work upon which you request our advice is represented by the following schedule compiled from the last estimate paid to the contractor:

4510.02 cu.yd. additional concrete @ $5.50	$24,805.11
18.76 cu.yd. concrete posts @ $9	168.84
322.81 cu.yd. cement coping @ $7	2,259.67
14,814.86 sq.yd. cement walks @ $1.60	23,703.78
9684.30 lin.ft. protection wall @ $22	213,054.60
414.50 lin.ft. Hollander's Creek drain at Broad St. @ $31	12,849.50
885.60 lin.ft. cement concrete steps @ $1	885.60
Total	$277,727.10

LIGHT PROTECTION WALL

The largest item of the work under this contract upon which our judgment is asked was a wall called a "Light protection wall." This wall is built to surround the lagoon, it rests upon piles, is 5 ft. high, from 2 to 3 ft. thick, and is surmounted by a rough stone top. The total amount paid for this wall was $213,054.60.

In a preliminary investigation before our first inspection a hole had been made in this wall. This was enlarged and six additional cuts were made through the wall and timber substructure with the necessary excavation to completely expose the substructure.

We formed our judgment of the character of the wall from an examination of the seven cuts above referred to, supplemented by an inspection of the whole length of the wall on both sides. We found about 1600 ft. of wall to be of poor quality not meeting the requirements of the specifications. The remainder, approximately 8080 ft., shows many defects, but considered broadly the concrete in it is sufficiently good so that it may be accepted.

For the concrete the contractor used an unwashed gravel, fairly well graded but full of impurities which tended to impair the quality of the concrete made from it. The gravel contained wood, lumps of clay, pieces of coal and some very fine material, part of which was organic matter. The presence of this foreign material accounts in part for the lack of coherence and hardness and poor surface in various parts of the work.

The use of this gravel was not permitted by the specifications, which required that broken stone should be used. As the broken stone would have been free from the impurities which were objectionable in the gravel, the resulting concrete would have been of better quality if it had been used. We are of the opinion that if the gravel had been properly washed and screened it would have been equal to the crushed

stone specified. The substitution of gravel for crushed stone therefore was not objectionable, but the use of gravel without proper preparation should not have been permitted.

The specifications permitted 5% of dirt or other impurities —and while we realize that this provision may be variously interpreted, we are of the opinion that this could not be construed as including wood, coal and lumps of clay. The specifications fix the maximum dimension of the aggregate as 1½ in. There were found throughout the concrete pieces ranging from ¼ to 5 in., and while we are of the opinion that for the mass concrete in the wall this was not objectionable, it was objectionable in the sidewalks where the base was only 4 in. thick.

In this defective concrete there are horizontal seams at points where there have been breaks in the construction of the wall as at the end of the day's work; and the concrete was not composed of sufficiently good materials and not sufficiently mixed to be of a quality to resist frost action. As a result of these defects fissures have already been produced which disfigure the wall.

FIG. 3. METHOD OF MAKING INVESTIGATORY OPENINGS IN WALL AND SIDEWALK

(Cut in wall has been made by investigators.)

This wall should have been built in monolithic sections with only vertical joints. It was not built in this way. Horizontal joints were also made and it appears that the seams are largely the result of failure to remove the laitance and impurities of various kinds, and to the concentration of the fine materials resulting from the excessive working of a wet mixture, which produced a layer of mortar of slow hardening quality at the top of the concrete placed at one time, and possessing insufficient strength to resist frost action or to permit the proper bond or union with the concrete subsequently placed.

The plans and specifications were defective in that there was no provision for joints for expansion and contraction of the concrete in setting and with changing temperatures. Notwithstanding this omission some joints were provided, but they proved to be inadequate, for many cracks have occurred at other points than the joints. The joints which were examined consisted of a layer of mortar averaging about an inch in thickness inclusive of tar paper and providing practically no compressive material. This paper had become hard and brittle and the joint was not effective. At a number of points inadequate mixing of the materials comprising the concrete was noted.

The contract required the walls to be topped with granite and we find that no granite was used, but instead gneiss, a cheaper material, was substituted.

The timber substructure did not comply with the requirements of the specifications, which provide that the piles shall be at least 12 in. in diameter at the top where cut off. It was found that in every section examined one or more piles of the four piles exposed failed to meet this requirement. One of the piles was only 7⅞ in. in diameter.

It was impracticable to lower the water in the lagoon sufficiently to expose the timber work; and the muddiness of the water prevented seeing the piles with sufficient clearness for measuring their diameter at other places than where cuts were made. While the number of piles examined is thus small in comparison to the whole number, nevertheless, we believe that the cuts at which the examinations were made, showing deficiency at each point, give a fair idea of actual conditions.

SIDEWALKS

A concrete pavement surrounds the lagoon and runs around the wall on and through the Island. In addition to this there is 388 ft. of pavement running southeasterly from the watch box. In order to determine the thickness and general character of this pavement, slabs were removed and examined.

The pavement generally has a width of 1 ft., made up of three rows of slabs each 4 ft. square. A surface examination was made of the slabs, 7604 in all, noting cracks, spalls or pitting of the surface. These surface pits were due to the presence of pieces of wood, coal and clay contained in the sand and gravel used for making the concrete. These had subsequently rotted or oxidized, or been washed out, leaving voids and pits in their places. These formed ugly holes, often an inch or more across and half an inch deep, and disfigured more than one-third of all the slabs in the pavement.

The surfaces of about 50% of the slabs examined we classified as good, that is, slabs with surfaces not seriously marred, but in no case did we find slabs that would represent first-class sidewalk construction, such as is readily obtainable for the price paid for this work. Eleven per cent. of the slabs were cracked and 35% were spalled.

In order to expose the timber sub-structure at the seven cuts in the wall, it was necessary to remove two of the pavement slabs at each point. In addition 17 other slabs were removed. In order to get a fair average of the whole work slabs were selected at 500-ft. intervals, so alternated as to give an equal number from the outside, center and inside courses, the latter being the one next the wall. The thickness of each slab was measured at 13 points, at the ends and center of each edge, and in addition, a hole was drilled through its center. In all, 403 measurements were made.

The specifications require the slab to be 5½ in. thick, consisting of 4 in. of base and 1½ in. of granolithic mixture. In our examination we did not find a single slab fulfilling these requirements. In all of the slabs the granolithic varies from ¾ to 2¾ in. and the base from 1 to 3½ in. The average of all measurements shows a thickness of granolithic mixture of 1.62 in. and of base of 2.55 in., making an average total thickness of 4.30 in. This is 22% less than the 5½ in. required.

In regard to subgrade, the specifications provide that:

> The walk shall be excavated and graded accurately to a subgrade of 5½ in., preparatory to paving; the contractor shall do all filling and leveling to the established grade, and soft and spongy places shall be dug out and refilled with coarse sand or gravel screenings, solidly rammed.

FIG. 4. SLAB OF CONCRETE SIDEWALK, SHOWING SAND SEAM SEPARATING GRANOLITHIC SIDEWALK FROM BASE

The sub-base under the 31 slabs examined was found to be ashes varying in thickness from 8 to 20.5 in., with an average of 14.4 in.

We are unable to find any provision in the specifications for this sub-base. It was unfortunate that ashes were placed in the subgrade, where the ground is saturated with water and no provision had been made for drainage. Frost action on this material saturated with water has apparently caused much of the movement and cracking that exists and which disfigures the pavement.

We are of the opinion that these sidewalks fail to fulfill the essentials for first-class sidewalks in the following particulars:

a. Inferior quality of sand and gravel used.
b. Lack of drainage.
c. Failure to provide for expansion and contraction by cutting the slab through.

d. Improper mixing and laying.

It is our opinion that these pavements cannot be satisfactorily repaired since the defective slabs intersperse the good throughout, and the foundation is not of a proper character. With the exception of a small portion laid southeast of the watch box, which is of better quality and workmanship and shows what was possible even under the conditions prevailing at League Island, the pavement is of inferior quality, not fulfilling either the requirements of the specifications or those of first-class construction. The pavements are so defective that they will have to be relaid.

BRIDGES

The island in the lagoon is reached from the main land by three concrete bridges each about 15 ft. wide, having two spans of about 20 ft. each.

The general plan shows three crossings, but does not indicate their character. The specifications make no reference to these crossings. So far as appears under the contract they might have been solid fills or timber or steel bridges. It appears that concrete arch bridges were built, although no plans for them have been found.

The work seems to have been paid for under items in the contract for "grading," "piles," "additional concrete posts," "yellow pine lumber," "cement coping," and "cement-concrete steps."

Under these conditions we are unable to determine whether these bridges comply with the requirements of the plans and specifications. We may state that they are examples of poor design and inferior workmanship.

As they were built into the wall about the lagoon, without provision for expansion and contraction, the pressure resulting from the expansion of this wall has resulted in spalling and cracking of the concrete in places and this disfigures them. The inferior character of the gravel, above referred to, made it impossible to obtain a satisfactory surface finish.

ENGINEERING

Our first thought under this heading was to speak of the lack of engineering ability shown by the design. On further consideration we are disposed to speak rather of the entire absence of engineering control. No proper design was made. The structures seem to have grown rather than to have been the result of definite design. For instance, we have been unable to find any design whatever for the bridges. The building of the so called light protection wall on piles which are practically the equivalent of stilts, and without providing expansion joints in the long lenths of it and especially at sharp bends and where it connects with other structures, does not suggest that any engineer had to do with it. The specifications were loosely drawn, inconsistent and lacking in essential details.

The conditions never justified the construction of the wall which is the principal item in the work regarding which you have asked our judgment, and it can serve no useful purpose. On the other hand, its presence is a serious handicap in the development of the park, and the greater part of it will have to be removed in order to provide the shallow water and beaches essential for a useful city park. The whole work and proceeding shows no sign of engineering control.

INSPECTION

The specifications provide for inspection as follows:

Inspector. The contractor will be required to employ at a monthly wage of $100 a month, an inspector to be named by the Department of Public Safety, etc.

This is a vicious provision which cannot be too strongly condemned. A city should provide its own inspectors. In this case the city, in undertaking work which costs hundreds of thousands of dollars, turned over the sole protection of its interests to a man employed and paid by the contractor. The fact that this man was nominated by an officer of the city afforded no protection as actual experience in this case fully demonstrates.

ADMINISTRATION

The manner of advertising for this work which ultimately involved the expenditure of over one million dollars, amounts to a scandal. The advertisement appeared in fine print in six local papers. Only nine days were allowed for the submission of bids. The proper interval would have been at least three or four times as long.

The advertisement begins with a request for bids for the construction of police, fire and patrol stations, thus giving the impression that the advertisement covered relatively small work. Afterward in five lines the work at League Island Park is mentioned and this is followed by other matter. Nothing is said to give bidders an idea of the nature or the magnitude of the work. For all information, bidders are referred to the City Forester, an individual unknown generally to contractors for work of this character. A deposit of 20% of the amount of the bid was greatly in excess of that usually required and tended to discourage or prevent competition.

We are decidedly of the opinion that work of this magnitude involving the expenditure of hundreds of thousands of dollars should have been most carefully planned and sufficiently advertised to secure competent bidders. Adequate time, at least several weeks, should have been allowed and full detailed plans and specifications provided to enable contractors to make intelligent bids.

The whole procedure which failed to give us [illegible] and proper publicity, suggests an attempt to comply with the law with the least possible publicity, in the desire to eliminate competition.

The contract covers much work in addition to the concrete work on which you have asked us to report. It was made for an original amount of $498,000 on May 21, 1909; it was afterward extended without inviting further bids and at the time work was stopped in February, 1912, there had already been paid to the contractor $995,123.24 or nearly twice the amount of the original contract.

Respectfully submitted,
ALLEN HAZEN,
RICHARD L. HUMPHREY,
FRED W. TAYLOR.

January 24, 1914.

Sewerage and Sewage-Treatment Systems and Financing, Vincennes, Ind.

The chief points of interest in the sanitary sewerage system completed at Vincennes, Ind., in November, 1913, are ventilation through the house sewers and soil pipe; disposal by dilution, with screening and multiple outlets, the latter to aid dispersion; the provision of an emergency hypochlorite disinfection plant; and the financing of the whole project by a private association. All these features of the sewerage system and franchise were described in a paper submitted to the Indiana Sanitary and Water Supply Association in the latter part of February, 1914, by Robert C. Wheeler, resident engineer for George W. Fuller, of New York City.

THE SEWERAGE SYSTEM

Vincennes has an area of 2.75 sq.mi., which varies not more than 13 or 14 ft. in level. The population of the city in 1910 was 14,895 and the assessed valuation was about $8,200,000. When sewerage construction was begun, in 1912, there were only 4 or 5 miles of sanitary sewers in the city, all privately owned.

The sewerage system as built comprises about 40 miles of 8- to 30-in. sewers, a pumping station about 1700 ft. below the city for use at high stages of the Wabash River, which receives the sewage; and the hypochlorite plant and submerged multiple outlets already mentioned. The plant is designed to serve a population of 30,000.

VENTILATION—Perforated manhole covers were prohibited because some of the streets are subject to flooding and because of the gravelly character of the street surfaces. Accordingly, the running traps in the house connections were omitted, giving ventilation through house sewers and soil pipes.

FLUSH TANKS—There are two hundred and sixty 250-gal. 6-in. automatic siphon flush tanks. Their water-supply "is controlled by flow-regulating devices each containing a hard-rubber disk with an orifice of such a size that under the average pressure in the water mains" the tank which it supplies will discharge once in 24 hours.

SEWAGE TREATMENT AND DISPOSAL

The Wabash River at Vincennes has a total dry-weather flow of about 2000 cu.ft. per sec., or about 11 to 12 cu.ft. per sec. per 1000 population for the 175,000

people in the cities and towns of the drainage area, including Vincennes.

No public water-supplies are taken from the Wabash River within 30 miles below the Vincennes sewage outfall. Nevertheless, it seemed desirable not only to screen the sewage but also to provide a hypochlorite plant for emergency use.

Screens.—There are two sets of bar screens, having clear openings of 7/8 in. and 1/8 in., respectively. The screenings will be burned in incinerators at the pumping station.

Hypochlorite Plant.—The screen and suction chamber give a detention period of from 1 to 15 minutes. There is a chemical mixing box, two chemical storage tanks with electrically driven agitators, an orifice box and a perforated pipe for measuring and distributing the solution, respectively. From 75 to 125 lb. of commercial hypochlorite per 1,000,000 gal. of sewage will probably be used.

River Dispersion by Multiple Outlets.—The outlet sewer is of 24-in. cast iron and extends about 300 ft. into the river. Five double outlets, 6 in. in diameter and about 30 ft. c. to c., discharge the sewage under water where the current is strong enough to prevent sedimentation.

Financing the Project by a Sewer Association

The estimated cost of the sewerage system was $344,000. For local reasons neither a bond issue nor assessments according to benefit were considered advisable as means of raising funds. The Vincennes Sewer Association was incorporated and was granted a franchise to build and operate sewers in accordance with plans and specifications which were included in the franchise. The city had the option of (1) paying an annual rental to the company for the use of the sewers; (2) buying its system at a decreasing price, stated in the franchise; (3) allowing the company to operate as a public utility and charge patrons a schedule of prices named in the franchise. In the first two cases the city was to bear the cost of operation and maintenance. The city selected the first or rental plan for the outset, at least. The Association issued 6% bonds to meet the cost of the system. The yearly rental is such that it will pay the interest on the bonds and retire all the bonds in 25 years. The purchase price, in case the city elects to buy, becomes less year by year, beginning with $334,675 on Jan. 1, 1914; falling to $241,400 on Jan. 1, 1924, and to $13,000 on Jan. 1, 1937.

Assuming that $5000 will cover operation, maintenance, insurance, etc., at the start, and adding this to $26,425, annual rental, gives a yearly charge of $31,425 for the year 1914, or a little more than $2 per capita. The results under this plan promise to be satisfactory.

The plans and specifications for the Vincennes work were prepared by Hering & Fuller, New York City, for the Board of Public Works. Construction was superintended by Geo. W. Fuller, of New York City, with Robert C. Wheeler as resident engineer—as already stated. The Vincennes Sewer Association let the contract for the work to Stewart Sheets & Co. Henry Watts, engineer for the contractor, gave lines and grades and drew record plans. Cost estimates, as a basis for payments to the contractor during construction, were made by Geo. W. Sturtevant, of Chicago, acting as engineer for the Association.

Finally, Mr. Wheeler states that while the engineering arrangement just outlined "was reasonably satisfactory at Vincennes, it would probably be advisable, as a rule," for a city to have more complete control of all the engineering operations.

Emergency Work at the Coffer-Dam of the Minneapolis Dam; Mississippi River

In all engineering construction work there are accidents and emergencies of major or minor importance, which (even when unimportant in themselves) afford valuable suggestions to men engaged in similar work. A case in point was a slip, on Feb. 6, at the coffer-dam for the "High Dam" now being built by the U. S. Government at a point between St. Paul and Minneapolis. This dam (dam No. 1, Mississippi River) will be 30 ft. high, with a lock giving access to a new harbor at Minneapolis, formed by the pool. The dam will back up the water just far enough not to interfere with the tail water from the St. Anthony Falls Water Power Co. Press reports were to the effect that a part of the dam itself had failed, but the "accident" was a slip of the coffer-dam, which was soon checked. For the following information, we are indebted to Lieut.-Col. Charles L. Potter, Corps of Engineers, U. S. A.

In the construction, each end of the dam was inclosed in a coffer-dam and the river allowed to pass between them. In one of these were built the lock and three dam sections; in the other were built the power-house foundation and ten dam sections, which sections contain eight sluiceways. The next step was to coffer-dam the river for the remainder of the dam, and to divert the water through the sluiceways and power house. To reach the sluiceways, it was necessary to raise the water above the dam about 2 ft., and in order to entirely carry the river, it was necessary to raise it to some head on the sluiceways.

The river is controlled at Minneapolis (about five miles above the new dam) by two dams which furnish power for flour mills, for electric lighting, and for street railways. The flour mills shut down from 8 a.m. Sunday to 8 a.m. Monday, which greatly reduces the utilized flow during that time. But they have little pondage capacity and can hold back the flow of the river less than one hour. A few miles above Minneapolis (at Coon Rapids) the Minneapolis General Electric Co. finished a dam about the middle of January. They had been taking advantage of the surplus waters to fill their pond on Sundays.

On Thursday, Jan. 29, the U. S. Engineer Office wrote the latter company that the coffer-dam was to be closed on Sunday, Feb. 8, and requested it to be in a position to hold (and to actually hold) on that day, all water not absolutely necessary for the street railway service. This necessitated the company using its steam plant for electric lighting. The company agreed to coöperate as desired, and so did the St. Anthony Falls Power Co., at Minneapolis. On Wednesday, Feb. 4, it was learned that, through some mistake in orders, the Sunday filling of the pool at Coon Rapids had gone on as usual on Feb. 1, and that the pond was absolutely full to the brim. The U. S. Engineers then had the work on the coffer-dam at such a point that they could not risk the company's emptying its dam. There could be no water held back by anyone on the following Sunday, and therefore Sun-

day would be no better than any other day. It was decided not to wait but to close as rapidly as possible, as a thaw might come and greatly increase the flow and the difficulties.

The river bed is a mixture of sand, gravel, shale with some clay and loam. It was decided to close as much could be dared (to the point where the heavier material would not move much) on Friday, and to finish the closure on Saturday. On Friday, about 4 p.m. (probably the hour of peak load at Minneapolis) the scour at the gap became sufficiently heavy to indicate that work could not be stopped for the night but must be pushed ahead continuously. As luck would have it, Friday night was the worst night of this winter, with sub-zero temperature, snow and heavy wind, a veritable blizzard. As the closure proceeded, the depth increased and a change was made from 32-ft. to 36-ft. sheet piling.

About 11 p.m., with only two or three piles to be driven, after one pile had been driven into about 4 ft. of solid material, the next pile, when dropped in place, went entirely through the waling pieces and floated down the river. A movement occurred in the coffer-dam and the piledrivers were withdrawn from the opening to the solid part of the dam. No further breach was made, but the point of intended closure was moved downstream about 6 ft.

The next morning, the coffer-dam was bulkheaded across each side of the part that moved, and the filling of the undisturbed portion has proceeded. The disturbed portion has been filled with rock, forming a dam which is now up to water level, and practically the whole river is going through the sluices and power house. The movement of material of the bottom was confined to the immediate point of the intended closure, and at 10 or 12 ft. upstream there was no scour. There was no ice movement involved.

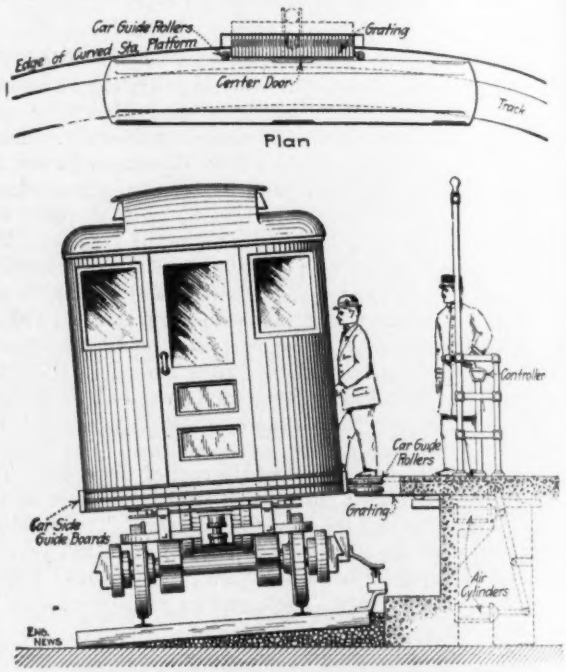
A patch will be formed in the disturbed section, the present coffer (with some planking) acting as the lower side, and a row of sheet piling about 15 ft. above, forming the upper side. This patch will give no difficulty in construction as the river is now diverted and the work will be in still water. It is stated that had it not been in the vicinity of two large cities, and had it not been necessary to give some publicity in the attempt to get the coöperation of the power dams above, "the matter would not have been mentioned in the press, and it would have been regarded as one of the circumstances which necessarily arise in the prosecutions of an engineering work of this size."

One peculiar thing happened. The point of intended closure moved downstream about 6 ft. and some of the sheet piling (triple-lap made of 3x12-in. plank) was tilted. A section of the coffer east of the opening, consisting of two rows of round piling with walings and two rows of sheet piling (driven in 4 to 8 ft. of river bed), moved bodily down river but remained vertical. It must have carried with it, by sliding, all of the river-bed material between the two rows of sheet piling. The theory is that it moved till it encountered a brush mat, heavily loaded with stone, which had been put in to prevent scour below the coffer. Whether or not there was this 4- to 8-ft. depth of material between the sheet piling when it moved, is not known certainly, as the engineers waited for no examinations the next day, till they had a good filling of stone in that part. Not less than 50 ft. moved bodily without getting out of plumb or opening the sheet piling.

Extensible Station-Platform Sections for the New York Subway

In ENGINEERING NEWS, Apr. 7, 1910, the great advantage was pointed out of movable platform extensions in increasing the safety of entering and leaving passengers in the New York City subways where some station platforms had been built on curves. It is obvious that with such platforms either the center or end doors of cars must be separated from the platform edge by a considerable length of step. To minimize the danger of accident, warning lamps have been placed under the edge of the platform, the train and station guards constantly call out "Watch your step," and there are numerous signs and annunciators giving the same warning.

A device of the type before discussed which has been experimentally tried out and found satisfactory at the 14th St. express station comprises a grating which slides out from the underside of the platform and up to the car door, together with a pair of air cylinders, operating levers, and control magnets as shown in the accompanying sketch. The general operation of the device is as follows. The platform guard takes position where the forward end of a train is expected to stop. Just before the first car comes to a stop, he moves a controller to the operating point when the magnetic valves admit air to the cylinders and the grated segment of the station platform moves out as the train comes to a stop. This platform segment moves in a level plane so that when it is moved outward it nowise affects the natural movement of the passengers. It remains out until the train has proceeded some seven or eight feet when it is crowded back by the car, which action reverses the electro-pneumatic cylinder control and causes the grating to be drawn further back and out of the way.



ARRANGEMENT OF MOVABLE GRATING FOR CURVED STATION PLATFORMS

A Service Fire Test of a Fireproof Building

Just before its completion in February last, the Amoskeag Savings Bank Building, a ten-story fireproof office building at Manchester, N. H., passed with conspicuous success through a service fire test of extreme severity. The building is a steel-frame structure with brick curtain walls and limestone trim. It is located on a corner and backs to an alley and the two street sides normally open to light are provided with plain glass windows. The fourth side abuts on another lot on which there was a four-story, stone-faced, brick-wall and timber-frame building of many years' standing. On this side the new building has a light shaft, but is built for its lower four stories in party wall with the old building, the upper six stories rising clear above the old roof. The entire ten stories in the shaft, and the six clear stories on the line wall are provided with windows with metal frames and sashes and wire-glass lights. The alley side is furnished with the same window equipment.

On Jan. 28, 1914, when the bank building was structurally complete, but while some interior work remained to be done, a fire broke out in the adjoining four-story building, which was occupied by a department store and was full of inflammable goods. The fire soon got beyond control and in the course of ten hours destroyed all the buildings on the block with the exception of the bank building, which not only was unharmed in itself, but also acted as an effective barrier against the spread of the conflagration to the remainder of the business district.

The wind drove the flames directly toward the bank building and for a number of hours they raged against the brick wall and wire-glass windows. When the fire was finally controlled, it was found that while some of the wire-glass panes were cracked, in no case was there any complete break or opening nor any damage to the sash frames. In examination for possible insurance damage, there was not a single sash that could not be raised with ease. The brickwork and terra-cotta cornice were somewhat blackened, but not otherwise damaged. During the whole of the fire, plasterers, tile layers, etc., remained at work in the interior of the bank building.

The accompanying view, taken after the fire, shows the ruins of the block, with the Amoskeag building in the background. The windows shown are the ones which were exposed to the flames.

A Modern Fireproof Building after Exposure to Conflagration. The Amoskeag Bank Building, Manchester, N. H.

The building was designed by Hutchins & French, of Boston, and was built by the R. H. Howes Construction Co., New York City. M. F. Westergren, Inc., New York, furnished the metal frame and sash.

A National Survey of Social and Industrial Progress is to be made by the National Civic Federation of New York City. The survey is to be under direction of Dr. Roland R. Falkner, who was Assistant Director of the U. S. Census in 1910. The purpose of the survey, according to the announcements sent out by the Federation, is to answer the question whether the social movements of recent years have been in the direction of economic justice and a larger human welfare, or the reverse. The subjects to be investigated by the Survey include such matters as profit sharing, workmen's compensation, industrial loans and savings, child labor, welfare work, social insurance, government ownership and operation of public utilities, and municipal government.

Collapse of Steel Grain Tanks, Austin, Texas

The accompanying halftone gives a view of the grain elevator plant of the Morten Milling Co., at Austin, Texas, where on Feb. 17, 1914, three of the large steel grain tanks collapsed following the initial failure of one of the three. The tanks which failed are the center ones of a battery of six of the same dimensions, that is, 26 ft. diameter, 70 ft. height and 30,000 bushels capacity. They were of standard steel tank construction, founded on a concrete base, footing into a new 15-ft. fill. The tanks had only recently been placed and the middle one in the rear of the view was in process of being loaded when marked signs of settlement were noted in the morning of Feb. 17. All workmen were ordered from the vicinity and when the collapse took place no one was in range of the falling material. The far middle tank first fell and carried with it two of the tanks in the foreground, both of which were empty. At the time of collapse there was about 25,000 bushels of wheat in the far middle tank and none in any of the others.

STEEL GRAIN TANKS WHICH COLLAPSED, DUE TO FAULTY FOUNDATIONS

We are informed that the engineering and contracting work was all done by a foreman employed by the company acting under office instructions. It is reported that the filled foundation was obviously insufficient for the load imposed and that there had been signs of settlement for some time previous to the collapse.

What Does a Subway Car Cost? The Public Service Commission, First District of New York, serving as a rapid-transit commission for New York City, has recently approved contracts for 100 new subway cars of the Brooklyn Rapid Transit Co. These cars are 67 ft. long and 10 ft. wide, compared with 51½ ft. and 8¾ ft. respectively for the existing subway equipment. The car bodies will cost $6119 each; the motor equipment, $1100; control apparatus, $975; brake set, $911; each truck, $450; total for each car, $10,005.

Conditions of an International Competition for Model Shops of the Chilean State Railways

The advertisement of an international competition for the plans and specifications of a central model workshop and four repair shops for the state railways of Chile, was very briefly noted in ENGINEERING NEWS of Feb. 12. Through the courtesy of the Chilean Minister at Washington, we are able to print the official statement of conditions of the competition as follows:

Under a decree of the Ministry of Railroads, dated Feb. 25, 1913, the basis for the contest is as follows:

(1) A first prize of $20,000 for the best plan and a second prize of $10,000 for the next best. Honorary mentions will be awarded also. The money prizes will be awarded only to plans for the Central Workshop.

(2) The plans and specifications should be received at the Ministry of Railroads, Santiago, Chile, not later than May 1, 1914.

(3) A Jury of Award composed of nine members appointed by the Government and presided over by the Minister of Railroads, will examine and pass upon the plans and specifications submitted, and report thereon within one month from the closing of the competition.

(4) The prizes will be paid within 30 days from the date of the jury's report.

(5) The Government reserves the right to reject any or all of the plans submitted, and consequently to withhold any or all of the prizes.

(6) Plans and specifications to which prizes shall have been awarded will become the property of the Government, and all others are to be returned to their authors.

(7) The fundamental requirements of the Central Workshop are as follows:

(a) A capacity for repairing 500 locomotives with tenders per year, provided that no large repair work shall take more than from 60 to 90 days of 8½ working hours. (An example of large repair work would be the changing of cylinders and boilers, and their accessories.)

(b) A capacity for the simultaneous and continuous repair of 50 passenger coaches and 400 freight cars.

(The present number of locomotives in operation is 600; number of passenger cars, 500; freight cars, 6000. These figures are to be increased every year, as follows: Coaches and cars, 10%, and locomotives, 5%.

(c) It should be so equipped as to meet all the needs of the general service, its administrative organization, and the machinery and plants.

(d) A capacity for producing not only the necessary pieces and parts, such as cylinders, boilers and mechanisms, but also such pieces as may be needed in the auxiliary repair shops.

(e) Adequate stores for the stocking of reserve materials.

(8) The Central Workshop may be presented in the plans as one plant to be built on a single site, or as two separate plants, one for locomotives and tenders, and the other for coaches and cars.

(9) As to the auxiliary repair shops, their capacity or importance shall be determined by each competitor, taking as a basis the Central Workshop and bearing in mind the requirements of the service and traffic.

(10) In drawing up these plans and specifications, the following facts should be taken into consideration:

(a) That the necessary sites are to be furnished by the Government without cost.

(b) That native materials are to be used in so far as possible.

(c) That prices of materials and machinery are to be estimated as placed on board at Valparaiso, and, therefore, not including custom duties.

(d) That the state railways will transport without charge all materials, but the cost of loading and unloading should be included in the estimates.

(11) The Government will furnish all available data and render all possible help in the drawing up of these plans and specifications, provided, however, that the competitor desiring such data or help shall have a duly authorized representative in Chile, who, on his behalf, shall express in writing his purpose to take part in the contest under these bases and designate the persons who as experts are to undertake the studies in Chile.

(12) This is exclusively a competition for plans and specifications, and the Government does not incur any obligation as to the future construction of the works.

Mid-Year Convention of the American Institute of Electrical Engineers

A year ago the American Institute of Electrical Engineers found the experiment of a mid-winter convention so successful that a second one was held Feb. 25-27 of this year and with nearly or quite equal success. About 350 registered. An attempt was made to have the sessions express the determination of the Institute to be the center of all professional activity of men connected in any way with the generation and use of electric energy, to make unnecessary the offshoot of special organizations devoted to special branches of electrical work and even to draw back under its wing those smaller societies which had already been formed. There was evident also the intent of the administration to have the Institute bring to its members all possible live information and experience in any way involving electrical energy.

The evening of the first convention day was given over to the Organization Committee of the International Electrical Congress (to be held in San Francisco in September, 1915), for nontechnical addresses to stimulate interest in this project. On the second night a dinner-dance at New York's newest hotel, the Biltmore, was given by the same Organization Committee.

The entire technical program was in the hands of the Electric Power Committee which, to carry out the spirit of the administration above noted, has been expanded into seven sub-committees (on stations, generation, protective apparatus, transmission, distribution, economics and data) each with its own chairman and from 5 to 27 members. Efforts seem to have been made to present, under each subdivision of electric-power topics, one rather general paper and one or more very specific ones.

At the opening session, after the Chairman of the Electric Power Committee, D. B. Rushmore, of Schenectady, had explained some of the matters referred to above, a report was received from the sub-committee on engineering data, P. H. Thomas, of New York, Chairman. It was explained that the committee was prepared to collect such live data of demonstrated utility as the members might call for and to publish it in such form as might generally be found most suitable. The committee put a number of questions before the members, that it might have all possible advice in shaping its policy.

The feasibility of an Institute handbook of electrical engineering was reported on by a special committee, of which Prof. Harold Pender, of Boston, was Chairman. From this report and the accompanying discussion, it was evident that electrical engineers generally were not satisfied with any of the handbooks which have been published. It was even stated that all of them combined would not form an ideal book. The report and the discussion, however, were decidedly unfavorable to the Institute engaging in such an enterprise.

There were four papers comparing the use of outdoor and indoor substations, by A. Macomber, of Boston, H. R. Fullerton, of Pittsburgh, F. L. Hunt, of Greenfield, Mass., and L. L. Perry, of Chicago. In general, the advantages of lower first cost, smaller fire hazard and simpler layout were found to exist with those designs which had transformers, oil switches, lightning arresters and busbar structures built for exposure to the elements instead of being housed. The disadvantages pointed out were difficulty of repair and uncertainty of operation under unfavorable weather conditions. For the very high-voltage sections of large substations and for all voltages on the very small stations required for certain rural communities the outdoor construction seemed to be favored in practice on grounds of economy.

The paper, "Protective Reactances in Large Power Stations," by James Lyman, A. M. Rossman, and L. L. Perry, of Chicago, described several schemes of inserting reactance coils in the main supply circuit to prevent excessive flow of current in time of trouble.

The report of the Transmission Sub-Committee, P. W. Sothman, Chairman, started out with a general review of the problems of high-tension transmission lines—such as character and size of rights-of-way, sleet troubles, good tower construction, aluminum, copper and copper-clad steel conductors, insulators, telephone interferences, etc. The second part of the report, in four appendixes, took up the deterioration of porcelain in service (by J. A. Brundige, of New York), extent of influence of a direct lightning stroke (by L. C. Nicholson, of Buffalo), transmission problems in the West (by P. M. Downing, of San Francisco), and switching (by G. Facchioli, of Pittsfield, Mass.). H. W. Buck, of New York, presented some notes on the practical operation of suspension-type insulators.

The report of the Sub-Committee on Distribution of Electrical Energy, P. Junkersfeld, of Chicago, Chairman, with nine similar appendices, presented a review of what

was considered good practice throughout the country and touched on some points in direct- and alternating-current work, in lighting and railway lines, etc., which were not settled or standardized.

One session was given up to the discussion of protective apparatus with papers on "Recording Devices," by C. P. Steinmetz, and "High-Frequency Relays," by E. E. F. Creighton, both of Schenectady.

C. T. Hutchinson, of New York City, presented a paper, "The Economical Capacity of a Combined Hydro-Electric and Steam Power Plant." A direct method was presented of handling this problem of supplementing seasonal deficiency in stream flow at a hydraulic development. The author showed a new form of presenting the stream characteristics by a "per cent.-deficiency" curve and the power-load characteristics by a "per cent-load" curve.

H. M. Hobart, of Schenectady, presented "The Cost of Electricity at the Source," which was a simple exposition of determining the cost of manufacturing electricity in a 100,000-kw. station with high load factor (50 to 100%) and with coal cost from $5 per ton down to zero. The author's figure for cost of installation at $35 per kw. of station capacity was criticized in the discussion; but he stoutly maintained that it represented the best practice which was to be expected from now on, and he subdivided this figure into $10.50 for generators, cables, etc., $8.50 for steam-raising apparatus, $5 for condensers, etc., $8 for buildings, and $2 for engineering and contingencies. In such a plant under variable conditions his assumptions and computations showed that electricity could be generated at a cost of 0.25c. to 0.40c. per kw.-hr., but it was emphasized that this was only a small part of the total cost of supplying a customer.

Break in a 36-In. Water Main at Springfield, Mass.

By Elbert E. Lochridge*

A break in a water main of the water-works of Springfield, Mass., occurred on Mar. 1, resulting in the partial wreck of a house by undermining.

To understand the local situation it should be known that the old distribution system of the Springfield water-works, constructed to receive water from the Ludlow system lying entirely east of the city, would not have been satisfactory for distributing water from the new or Little River system, which was put in service four years ago, and which lies west, across the Connecticut River from the city. To properly connect all portions of the city with the new supply, a circuit main 7½ miles in length was built encircling the city and passing through each section. The pipe composing this circuit was 36, 30 and 24 in. in diameter. The portion of this main to and through the business section is in Water St. and is 36 in. in diameter, and it was on this main, near Vernon St., that the trouble occurred early Sunday morning, Mar. 1.

A drop in pressure from a normal of 144 lb. to 120 lb. was noted on the recording gage at the water-works office. Several minutes later a patrolman's telephone reported the location of the trouble. The pressure on the recording gage remained steadily at 120 lb. from 12 until 1:05 a.m., when a second sudden drop to 70 lb. occurred, and the pressure remained near this point until 1:50 a.m. when the normal pressure was restored.

The first effect of the water was to enter the cellar of a brick house, on Water St. The main at this point is on the east side of the street, but a short distance out from the curb and the house stood close to the walk. The final result was the collapse of half of the house into the basement.

To control the water at this point it was necessary to close two 24-in. gates on the 36-in. main and also the gates on the seven side streets lying between Main and Water Sts. The snow and slush prevented as rapid work as could have been done at another season of the year, but all of these gates were closed in 90 minutes from the first indication of the gage that a break had occurred and the pressure restored to all of the business section except the portion of the street immediately affected.

Two hours later, a little before 4 a.m., a serious fire broke out in the Park Theater, two miles distant. The pressure chart indicated, however, that although 14 fire streams were used for an hour and a half, at no time was the pressure drawn below 140 lb. This would demonstrate the value of a circuit which permitted the flow of water from both directions even though one section was disabled.

An examination after daylight indicated that the damage was done entirely by the jet of water from the 8-in. line which supplied a hydrant at this point, and no portion of the pipe or hydrant have as yet been found. The nearness of the pipe to the building brought about a very early collapse and the damage was all done soon after the first rush of water. There were four families in the house, all of whom made their escape with little time to spare.

This portion of the line has been in service nearly eight years. The 36-in. line, although laid bare by the rush of water, was not undermined or affected. The frost had extended about 4½ ft. into the ground and this formed a solid cake over the point of leakage. What portion of the damage could have been caused by the change in position of this cake, due to a small leak from this or some other service, is entirely unknown at the present time.

Within twelve hours two other side connections in the city in widely separated lines were also broken from the mains, causing damage of lesser account. No estimates have been made as to what the total damage will be.

*Chief Engineer Water Department, Springfield, Mass.

NEWS NOTES

False Reports of a Tunnel Accident were quite generally published by the morning newspapers of Mar. 2. It was stated that on Mar. 1 the steelwork at the portal of the Baltimore & Ohio R.R. tunnel, now under construction, collapsed after a heavy blast, burying seven workmen. The contractor for this work is H. S. Kerbaugh, Inc., of New York City. According to an official statement a fall of rock on Feb. 26 slightly injured one man.

The New House of the Institution of Mining and Metallurgy, at 1 Finsbury Circus, London, England, was formally opened by the Lord Mayor of London on Jan. 13, which was the 22d anniversary of the foundation of the Institution. The arrangement of the house is as follows: Basement, smoking room, etc.; first floor, council chamber and reading room; second floor, library; third floor, writing rooms, etc.; fourth floor housekeeper's apartments.

A New Passenger Terminal at Buffalo is to be built by the Lackawanna R.R. The annual report of President Truesdale, just published, says:

The most important work the company has planned for the coming year is the construction of new passenger terminal facilities at Buffalo. These facilities are imperatively needed, and negotiations have been under way for the past five years with the authorities of the city of Buffalo for the necessary rights and privileges in certain streets in that city, the use of which was required in order to carry out the company's plans. The expenditure on this account will be large, but the management deems that it will be justified, in view of the company's large passenger traffic to and from that important city.

Heavy Storm Damage in the Northeastern States occurred on Mar. 1 and 2. Railroad, street railway, telephone and telegraph services generally, were stopped. As an example of interruption to railroad traffic, no Pennsylvania R.R. trains reached the Thirty-third St. Station, from 7 p.m. on Mar. 1, until noon on Mar. 2—a period of 17 hours. Snow fell to a depth of 14½ in., the first 6 in. was very wet and caked easily. The later snowfall was of light consistency, easily drifted. The wet snow formed around overhead electric wires to a diameter of from 1 to 2 in. This condition, coupled with winds of from 60 to 84 miles per hour, caused unprecedented damage to electric-light, telephone and telegraph systems in the section affected. Sixteen persons are reported dead as a result of the storm.

Bonds for the Moffat Tunnel Project, described in "Engineering News," Jan. 22, 1914, were approved by the city of Denver, at a municipal election held on Feb. 17. The vote was 12,640 to 6953. It is reported that work on the engineering plans has already started, that field parties within a couple of weeks will take topography at the approaches, and that contractors may be expected to begin work in two or three months. It is planned first to drive a bottom heading, 7x12 ft., in from each portal for about 1000 ft. while power plants are being built and other equipment assembled for the bench excavation.

Action to Compel Water Purification at West Reading, Penn., was recently taken by the Pennsylvania Department of Health, through Dr. Samuel G. Dixon, commissioner. We are indebted to F. Herbert Snow, chief engineer of the Department, for the following statement of the case:

The West Reading Water Co. supplies a growing community opposite the city of Reading on the west bank of the Schuylkill River. The water is drawn from the river at a point below numerous industrial plants and a storm sewer outlet, and is totally unfit to use for human consumption without filtration. The filter plant is old, out of repair and totally inadequate. The company has been required by the State to treat the water with a germicide and the Commissioner of Health ordered that a new filter plant be installed. But the company delayed for various reasons the preparation of the plans and finally the Commissioner of Health put the matter into the hands of the Attorney General, with the conclusive evidence that the company had failed to fulfill its charter obligations in that the water it was supplying to the public was not pure and wholesome, after repeated admonitions from the Commissioner of Health to this effect. George A. Johnson, of New York City, is now in the employ of the company and an entirely new and uptodate plant will be installed at once.

The Tennessee River Bridge Competition at Chattanooga on Feb. 18 was attended by representatives of ten different engineering firms as follows: Brenneke & Fay, of St. Louis; Daniel B. Luten, of Indianapolis, represented by the Ellis Soper Engineering Co., of Chattanooga; W. C. Spiker Engineering Co., of Atlanta; Hedrick & Cochrane, of Kansas City, Mo.; C. E. Shearer, Memphis; John Ash, Chattanooga; Wilbur J. Watson & Co., Cleveland, represented by the Edward E. Betts Engineering Co., of Chattanooga; Waddell & Harrington, Kansas City; B. H. Davis, of New York. The Scherzer Rolling Lift Bridge Co., of Chicago, had a representative who appeared before the Commission. The Strauss Bascule Bridge Co., of Chicago, as consulting engineer for the vertical lift span, also had a representative.

The presentations of the designs offered by the different firms continued for two days. The Chattanooga "Times" says:

Engineers last night stated that they had never been accorded a more courteous and patient hearing than was granted them by the local commission. No engineer was able to complete his presentation of plans in less than a half hour and some consumed more than two hours.

The plans and propositions submitted included from one to four sets of plans and suggestions of designs for the bridge, a general proposition for the work and explanations of bridges they had built at other places, along with data on the bridge question, which has filled one room in Engineer Bryan's office. These will be held by the commission for their information until they have selected their engineer, when they will be returned to the engineer presenting them.

The War Department has limited the commission in many respects. One requirement of the department is that the spans over the river must be of not less than 300 ft. each. If this is required it will be necessary to build a steel bridge. If the department will permit spans of less than 300 ft. it is possible for a concrete structure to be built. The department has also specified that the bridge must be 100 ft. above low water. If this is adhered to it will be necessary to build a lift bridge. However, two engineers presented plans of bridges they had built by special permission of the government that were only 75 ft. above low water.

Each of these provided for a lift being built later should the government require it. From what can be gathered this is the bridge preferred, unless a concrete bridge is selected. The commission of each engineer was the same, 5% of the total cost of the bridge.

On Feb. 24, the Bridge Commission decided to postpone indefinitely the selection of an engineer, as stated in the "Times" of Feb. 25:

The Market St. bridge commission yesterday in executive session decided to postpone indefinitely the selection of a chief engineer for the bridge. Following this announcement the engineers who submitted plans and who have been in the city for a week pushing their claims, left for their respective homes. The commission will attempt to go over the matter alone with the government for the present and leave the selection of an engineer for a later consideration.

PERSONALS

Mr. John W. Everman, General Superintendent of the Texas & Pacific Ry. at Dallas, Tex., has been elected President of the Union Depot Co. of El Paso, Tex.

Mr. R. A. McCranie, District Superintendent of the Atlantic Coast Line R.R., at Waycross, Ga., has been promoted to be General Superintendent at Savannah, Ga.

Mr. F. J. Walsh, recently Superintendent of Construction for the Northwestern Electric Co., Portland, Ore., has been appointed Chief Engineer of the Port of Astoria, Ore.

Mr. W. E. Truesdell, formerly Consulting Engineer of 7 W. 38th St., New York City, has become a member of the firm of Joseph H. Wallace & Co., Industrial Engineers, of New York City and London.

Mr. F. W. Leatherby has been appointed Division Engineer of the Gulf, Colorado & Santa Fé Ry., with headquarters at Temple, Tex., succeeding Mr. F. D. Griffin, who has been placed in charge of construction of the new union terminal at Dallas, Tex.

Mr. J. T. Taffany, Division Engineer of the Erie R.R., at Buffalo, N. Y., has been transferred to the operating department as Trainmaster at Buffalo. Mr. Taffany is succeeded as Division Engineer by Mr. Frank S. Wheeler, Supervisor at Deposit, N. Y.

Mr. Robert M. Feustel, of the firm of Sloon, Huddle, Feustel & Freeman, of Madison, Wis., has been appointed Chief Engineer of the Illinois Public Utilities Commission. Mr. Feustel was for some time with the engineering staff of the Wisconsin Railroad Commission.

Mr. Barrett Smith, a mechanical engineering graduate of Cornell University, class of 1904, in charge of the advertising of Stone & Webster, Boston, Mass., has opened an office at 20 Central St., Boston, for the general practice of publicity and advertising work.

Mr. Carl R. Gray, whose resignation as President of the Great Northern Ry. was noted in our issue of last week, is now said to have been selected to be President of the Western Maryland R.R. to succeed Mr. J. M. Fitzgerald. It was previously reported Mr. Gray would become President of the St. Louis & San Francisco R.R.

Mr. Harry B. Hommon, recently Chief Chemist and Bacteriologist at the sewage-testing station, Cleveland, Ohio, has been appointed Sanitary Chemist in the United States Public Health Service to assist in the investigations recently authorized by Congress to study stream pollution by sewage and industrial trade wastes. Mr. Hommon, during the last ten years, has had a wide range of experience in treating industrial trade wastes and studying their effect on rivers at the testing stations at Columbus, Ohio, Waterbury, Conn., Gloversville, N. Y., Chicago, Ill., Akron and Cleveland, Ohio.

Mr. Thomas R. H. Daniels, Assoc. M. Am. Soc. C. E., has resigned as Chief Engineer of the Birmingham Railway, Light, Heat & Power Co., Birmingham, Ala., and is now Engineer of the Terre Haute, Indianapolis & Eastern Traction Co., Indianapolis, Ind., of which he was Assistant Engineer from 1909-1911. Mr. Daniels began electric-railway work in 1898 as Assistant Engineer in charge of track and roadway of the Rhode Island Co., Providence, R. I. Subsequently he spent two years with the engineering staff of Westinghouse, Church, Kerr & Co., New York City, and two years as Assistant Engineer in charge of rebuilding the street-railway tracks of San Francisco, Calif., after the earthquake of 1906.

Mr. H. S. Dickey, President and General Manager of the Dickey Construction Co., South Bend, Ind., recently sailed from New York City for San Domingo, W. I., as Special Representative of the United States State Department. Mr. Dickey is to make an appraisal of all the property built and

building by the San Domingo Department of Public Works, including steam and electric railways, highways, bridges, electric power plants, docks and lighthouses. This work will require a personal inspection of the works and the visiting of practically every city and town in San Domingo. Mr. Dickey spent nearly 25 years in railway location, construction and operation work in the Middle West, before he organized the Dickey Construction Co., about two years ago.

Mr. Alfred B. Fry, M. Am. Soc. C. E., Chief Engineer of the U. S. Treasury Service, Member of the Board of Consulting Engineers, Improvement of New York State Canals, has been appointed Consulting Engineer of the Department of Water Supply, Gas and Electricity of the city of New York, succeeding Mr. Edward Wegmann, M. Am. Soc. C. E. Mr. Fry has been with the Treasury Department since 1886, and has had experience in connection with the design, operation and construction of steam, electric and hydraulic power plants and of federal building work in Washington, D. C., Boston, Mass., Chicago, Ill., San Francisco, Calif., and other of the large cities. Since 1903 he has also been detailed for additional duty as Chief Engineer of the United States Immigration Service.

Mr. Edward Wegmann, M. Am. Soc. C. E., has been removed as Consulting Engineer of the Department of Gas, Water Supply and Electricity, after 30 years' service with the city of New York, and has been appointed General Inspector of the department at a salary of $3000 per annum. Mr. Wegmann was graduated in civil engineering at New York University in 1871, and his first engineering experience was as axman and chainman on preliminary surveys for the New York, West Shore & Chicago R.R. After two years on railway-construction work in the East he spent some months at the Wyandotte Rolling Mills in Michigan, studying iron manufacture. In 1875 he returned to Brooklyn, N. Y., where for a time he was a partner of Robert Creuzbaur in the manufacture of steam engines, later serving as Mechanical Engineer at the Danforth Locomotive Works, at Paterson, N. J., in superintending the construction of machinery under Mr. Creuzbaur's patents. In 1877 Mr. Wegmann returned to civil engineering as Engineer for the Keystone Bridge Co. in charge of erection work for part of the Sixth Ave. Elevated R.R. in New York City. The two following years he was Assistant Engineer with the Metropolitan Elevated Ry., New York City. From 1880 to 1884 he was in railway-construction work, first with the New York & New England R.R., and later with the New York, West Shore & Buffalo Ry. Since 1884 Mr. Wegmann has been in the municipal engineering service of the city of New York. His first appointment under the old Aqueduct Commission was that of Assistant Engineer of Construction in the office of the Chief Engineer. Subsequently he was Division Engineer in charge of construction of various divisions of the Croton Aqueduct. In 1904 Mr. Wegmann was appointed Expert Engineer of the Aqueduct Commissioners, and in 1907 Consulting Engineer. He was Chief Engineer of the commission in 1910 until it was abolished; since then he has been Consulting Engineer of the Department of Water Supply, Gas and Electricity, with a salary of $5000 per annum. Mr. Wegmann is the author of a history of the New York City water supply and the well known "Design and Construction of Dams."

OBITUARY

Robert Kennedy Duncan, Director of the Mellon Institute of Industrial Research of the University of Pittsburgh, died at his home in Pittsburgh, Penn., Feb. 16. He was born in Canada in 1868 and graduated at the University of Toronto in 1892. Subsequently he studied at Clark University, at Columbia University and in Europe. Prof. Duncan was a well known contributor to the newspapers and periodicals, and he was the author of the books "The New Knowledge," "The Chemistry of Commerce" and "Some Chemical Problems of Today."

Charles H. Strong, founder and former head of the contracting firm of C. H. Strong & Son, Cleveland, Ohio, died from apoplexy, Feb. 21, at the home of his daughter in Cleveland. He was born in 1831 in what is now East Cleveland, and he early took up engineering work. In the eighties he was City Engineer of Cleveland. Later he became a railway construction contractor and built what is now a part of the Wheeling & Lake Erie R.R. Mr. Strong is survived by a widow, to whom he was married in 1861, and two sons, C. H. Strong, Jr., Vice-President of Wm. Taylor Son & Co., and H. B. Strong, head of the firm of C. H. Strong & Son, Contractors, both of Cleveland.

John Whitney Barlow, Brigadier-General Corps of Engineers, U. S. A., retired, died Mar. 1, in Jerusalem, Palestine. He was born at Perry, N. Y., in 1838 and graduated at the United States Military Academy at West Point in 1861, at the beginning of the Civil War. He was at once commissioned Second-Lieutenant in the Artillery Corps and sent to the front. Nine days later, Lieut. Barlow was transferred to the Corps of Topographical Engineers and a year later to the Corps of Engineers. He saw active service within a few days after receiving his first commission at the first battle of Bull Run in Virginia. Later he was with Gen. Gen. B. McClellan in the Peninsula campaign and was thrice breveted "for gallant and meritorious services." Subsequently he served with the engineer battalion of the Army of the Potomac in charge of the work of building bridges, making roads, building block houses and erecting defensive works. He was chief of the 17th Army Corps in charge of the defenses of Nashville, Tenn., 1864 and 1865. After the war he superintended the building of Fort Clinch in Florida and Fort Montgomery, New York. He was commissioned Major of Engineers in 1869 and Lieutenant-Colonel in 1884. He was in charge of harbor work at Chicago, New London, Milwaukee, Chattanooga and Nashville and made some notable explorations of the headwaters of the Yellowstone and Missouri Rivers in what is now the Yellowstone National Park. He was Senior Commissioner for the United States in marking the international boundary between the United States and Mexico during 1892-1896. In 1895 he was commissioned Colonel of Engineers and was appointed Division Engineer of the Southwest and Northwest divisions, successively, of the United States Engineer Department. He was commissioned Brigadier-General and Chief of Engineers on May. 2, 1901, but the following day was retired at his own request, after 40 years' service. He is survived by a widow whom he married in 1902.

ENGINEERING SOCIETIES

COMING MEETINGS

ILLINOIS WATER-SUPPLY ASSOCIATION.
Mar. 9-11. Annual meeting at the University of Illinois. Secy., Edward Bartow, Urbana, Ill.

VERMONT SOCIETY OF ENGINEERS.
Mar. 12. Annual meeting at Burlington, Vt. Secy., Geo. A. Reed, Barre, Vt.

CENTRAL RAILWAY CLUB.
Mar. 12. Annual meeting at Buffalo, N. Y. Secy., H. D. Vought, 95 Liberty St., New York City.

ARKANSAS ENGINEERING SOCIETY.
Mar. 16-17. Annual meeting at Pine Bluff, Ark. Secy., P. B. Hill, Little Rock, Ark.

AMERICAN RAILWAY ENGINEERING ASSOCIATION.
Mar. 17-20. Annual convention at Chicago, Ill. Secy., E. H. Fritch, 900 S. Michigan Ave., Chicago.

HIGHWAY ENGINEERS' ASSOCIATION OF MISSOURI.
Mar. 18-20. Annual meeting at St. Joseph. Secy., Ray L. Gargill, St. Joseph, Mo.

EFFICIENCY SOCIETY.
Apr. 4-11. First annual conference in New York City. Offices at 41 Park Row, New York City.

AMERICAN ELECTROCHEMICAL SOCIETY.
Apr. 16-18. Spring meeting in New York City. Secy., Prof. J. W. Richards, Lehigh University, Bethlehem, Penn.

AMERICAN DRAINAGE CONGRESS.
Apr. 22-25. Annual meeting at Savannah, Ga. Secy., J. R. Gray, Chamber of Commerce, Savannah, Ga.

IOWA STREET AND INTERURBAN RAILWAY ASSOCIATION.
Apr. 22-25. Annual meeting at Waterloo, Iowa. Secy., H. E. Weeks, Tri-City Ry. & Light Cos., Davenport, Iowa.

Iowa Engineering Society—At the annual meeting, held at Council Bluffs, Iowa, Feb. 17-18, the following officers were elected: President, Prof. W. G. Raymond, Iowa City; Vice-President, W. H. Kimball, Davenport; Secretary, S. M. Woodward, Iowa City.

Arkansas Engineering Society—The third annual meeting of the Society will be held at Pine Bluff, Ark., Mar. 16-17. There will be a banquet on Mar. 17. Major W. J. Parkes, of Pine Bluff, is President. The Secretary is T. B. Hill, of Little Rock, Ark. The chairman of the entertainment program committee is Frank R. Allen, City Engineer of Pine Bluff.

Ohio Engineering Society—At the recent annual meeting held at Columbus, Ohio, officers were elected as follows: President, F. M. Kennedy, Washington C. H., Ohio; Vice-President, Wm. F. Schepflin, Fremont, Ohio; Secretary and Treasurer, John Laylin, City Engineer, 712 Hartman Building, Columbus, Ohio.

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